THE ENGINEERING RECORD

PUBLISHED WEEKLY BY THE

McGraw Publishing Company

James H. McGraw, Pres.; Curtis E. Whittlesey, Sec. and Treas.

MAIN OFFICES:

239 WEST 39TH STREET, NEW YORK.

BRANCH OFFICES:

CHICAGO: Old Colony Building.

PHILADELPHIA: Real Estate Trust Building.

CLEVELAND: Schofield Building.

SAN FRANCISCO: Atlas Building.

LONDON OFFICE: Hastings House, Norfolk St., Strand.

SUBSCRIPTION RATES.

United States, Cuba and Mexico.....$3.00 a year
Canada .....$4.50 a year
Foreign subscription.....$6.00 a year
25 shillings. 25 marks. 31 francs.
Foreign subscriptions may be sent to our London office.

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Changes in advertisements intended for a particular issue should reach the office of THE ENGINEERING RECORD on Friday of the previous week. New advertisements can be received up to noon of Wednesday of the week of issue.

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Entered at the New York Post-Office as Second Class mail matter.

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Of this issue of THE ENGINEERING RECORD 15,000 copies are printed. Total circulation for 1907 to date 748,000 copies, an average of 14,667 copies per week.

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Cost Estimates in Hydro-Electric Transmission.

It is doubtful if any subject related to the production and sale of power for commercial purposes deserves more careful consideration at the hands of engineers than cost estimates in hydro-electric transmission. The reasons are not far to seek. Power distribution from steam plants is on the whole more restricted than transmission from hydraulic stations; the real question is narrowed down as a rule to securing the best location with respect to the market that is consistent with a given cost of land per square foot. The market itself lies within a smaller area, with rare exceptions; and the first cost, operating expenses and success of the selected equipment in service introduce few uncertainties unless standard types of apparatus are forsaken. There are, of course, notable exceptions to the prevalence of relatively narrow markets for coal-generated power in specific stations, such as the field of service of the Boston Edison Company's main station, or the distribution area of the Union Traction Company's central plant in Indiana. But in general there is little doubt that the commercial steam plant production is associated with more readily verifiable cost data than are available for hydro-electric estimates.

In the transmission of power from hydro-electric plants the competition of coal at distant markets, scattered over many counties, perhaps; the reliability of many miles of line operated at very high voltages; the fluctuations in the water supply at the generating station, the cost of flowage rights and of land for the transmission right-of-way, all complicate the estimated expenses and profits of the enterprise. Uncertain as these factors sometimes are they must be handled with the most unsparing frankness in preparing a sound estimate, and on top of all comes the vital question of development cost. To assume that a water-power project is a synonym for financial success simply because coal is eliminated from the generating process and fewer men are required at the plant per kilowatt of capacity, is the height of rashness. Recent observation of the commercial conditions in a number of hydro-electric plants confirms the conviction that the cost of development is second in importance to no other factor in the success of a modern transmission, on the monetary side.

Haphazard assumptions that the flow of a river as observed in one or two years can be counted upon indefinitely are likely to lead to serious expenditures for auxiliary steam equipment, especially in locations unfavorable to the storage of water in reservoirs. In proportion as records of flowage are in existence—and by records are not meant the stimulated recollections of the long bearded cross-roads grocery parliament—covering many years of extreme and average conditions, so will be the accuracy of the capacity estimate. So intimately is the development cost bound up with the available flow at different seasons in relation to head and storage, that long and painstaking checking and studying of the stream's discharge is amply justified, even though it increase the engineering cost by many hundred dollars.

Failure to utilize to the utmost every favorable natural condition in the hydraulic development exacts its tax in increased fixed charges. The estimate which overlooks the possibility of storage capacity for even 24 hours' operation falls short of the work. It costs money to keep a man on duty constantly and the boilers banked at an auxiliary steam plant, and where this is the practice, it is decidedly worth while to sit down and figure every possible scheme to secure one or two days' storage, at the least. It is not enough to make an estimate of the hydraulic development based on one or two hasty trips into the watershed to be utilized, and refreshed by hearsay data as to the flowage and drainage characteristics of the locality. Provision for the growth of the plant also receives scanty consideration in some reports. The cost of labor, of materials, expense of food supplies and of transportation may be lumped in a round figure of plant cost per kilowatt of capacity, but unless the local factors in these questions are known, uncertainty is given the chance to increase the fixed charges, which are as a rule the most important sum in the total cost of water power per unit of output.

Once the hydraulic development cost and probable average and minimum power available are carefully estimated, the way clears to a discussion of the rate problem. To a large extent rate consideration must be tentative in hydro-electric estimates, for under the stress of commercial operation and developing competition the rates may be obliged to change their shape if not their substance. Failure to estimate the minimum rate which, on the average, will pay the requisite operating expenses and investment charges, including depreciation of hydraulic as well as electric structures, invites disaster. There is little doubt that the charges of not a few hydro-electric power companies are too low, in consideration of the total cost of power production. Thus, to generate current in a hydraulic plant requiring the services of a steam auxiliary constantly as a reserve and often as a supplement, to transmit it 25 miles and sell it at from 3 to 1.5 cents per horse-power-hour even to consumers of moderate motor capacity, is a doubtful policy on the score of reasonable profit, even with coal at $6.00 per ton. In the particular system in mind which does this, it would be a difficult matter to raise the rates, for the present consumers never accurately figured the cost of their steam power before it was superseded by electricity, and were accustomed to paying power costs not in a single monthly bill, but in scattered items. If the original owners of the plant had made or purchased the necessary accurate cost estimates before embarking on the enterprise, the conditions to-day would be more favorable to securing the profit which could rightfully be expected in addition to the present divided rate.

The Policy of Secrecy.

An address by Dr. James Douglas, published on Dec. 7 in these columns, sets forth with admirable breadth of view the desirability of free interchange of ideas throughout the techincal world. Time was when, in all technical processes, profound secrecy was the order of the day. Patents were in ill odor, and the measure of protection secured by them was judged inadequate, so that when a valuable process was discovered it was worked behind closed doors so long as practicable and employees were muzzled as effectively as possible. The obvious difficulty of keeping many trade secrets against acute detective work and bribery has done more than anything else to check the practice, and while trade secrets still persist they are generally now of relatively minor moment, the really important advances being made the subject of patents.

Were the patent law more perfect and comprehensive, secrets would practically disappear and technical workers all the world over would live in a freer atmosphere. But the fact is that under the present rules of patent practice there are some things which are very difficult to protect adequately and others which can hardly be protected at all. One of the curious results of this is the not very infrequent use of a singular combination of patents and secrecy, a very generalized process being patented, *in terrorem* as it were, while the real process hidden behind it is practically a trade secret. In theory a patent must fully disclose to one skilled in the art the subject matter involved, so that the process could be actually worked or the apparatus successfully built on the basis of the published patent. As a matter of fact, and in the face of existing decisions, the nature of a technical disclosure is somewhat hazy and many patents are granted in which the disclosure is so incomplete that the real invention as practiced in very difficult to dig out of its environment. For example, if one looks up the patents on smokeless powder some very extraordinary misinformation can be se-

cured. Some of them are clear enough, but in others the list of alleged components and proportions is so vague that no one except the putative inventor could go ahead and make the supposed compound without a research quite equivalent to an independent invention. Some dye-stuff patents are almost equally bad, and it is reported that in the well-known case of the chemical final exhaustion of incandescent lamps one of the important defenses is incompleteness of disclosure. It is commonly said, too, that the data in some patents on photographic lenses are such as to require entirely independent redesign in order to produce anything of value.

In other cases, while the process of a patent may be technically fully disclosed, details necessary to commercial success are suppressed. All told, the number of commercial processes thus worked by practical secrecy although nominally patented, is very considerable. There are, besides, a good many instances of secret processes which, so far as anybody knows, are quite unpatented. Some few remain in the steel industry, in various metallurgical works, in the manufacture of special kinds of glass, and elsewhere. There is little doubt that in some cases secret details are important and valuable, and that the present patent system would not adequately protect them. Patents upon process are singularly hard to get in satisfactory shape, owing to various quantitative relations. In metallurgy, for instance, a particular reaction may give first-class yield only with particular proportions of ingredients or under peculiar temperature conditions. Infringement would be almost impossible to prove, even if the detail necessary to practical success could be patented, which is often difficult or impossible. The courts now distinctly recognize property in trade secrets and are disposed to protect it from theft or from improper use, as explained in previous issues of this journal.

Granting, then, that there may be wisdom in the preservation of trade secrets, one must admit that few technical manipulations really rise to that dignity. In most cases, the few little "wrinkles" held in a factory could be with great advantage swapped for others held somewhere else. It is one of the excellent functions of technical organizations to serve as a sort of clearing house for just these things. On the whole, and in the long run, a dog-in-the-manger policy does not pay either individuals or corporations. The engineer who freely and cheerfully gives helpful casual information on technical subjects to his colleagues makes more friends and gets more business than the solemn-faced chap who looks wise and hints grimly at important secrets when asked to lend a hand.

Of course an engineer, while developing a new apparatus or method, must in self-protection keep much to himself, lest some one with larger resources take the cue and push ahead of him. And yet stolen inventions are rare outside of novels—at least in the sense of inventions purloined prior to completion and developed for the benefit of the thief. There was a case of this sort a couple of years ago in which the inventor's mechanician queered an experiment and afterwards worked it out with some wicked partners and then sued the original inventor on the resulting patent. But the court arose and smote the offenders, and one can say generally that such larceny has very seldom been successful. In fact more inventors have suffered from failing to make disclosures of their work to their friends than have ever found disclosures misused. It is well, therefore, to take the optimistic view of human nature and to realize that one rarely meets a confrere who has nothing to give or is unwilling to impart what he has. One reason for the rapid progress of the arts and sciences in the last century was the growth of technical organizations and literature and the spread of a certain feeling of brotherhood among technical men.

At the present time there is more free and kindly interchange of information than ever before, and one has a right to expect still greater results in science and the arts. The technical papers read before various societies and the following discussions are rich in facts that a generation ago would have been jealously guarded, by which policy progress, which after all makes for general success, would have been checked. One kind of information, to venture a suggestion of improvement, ought, however, to be more freely given than is usual—and that is the results of failures. It takes a rather big man to own up to the futile things which he has attempted and yet such confession should be received with honor, for it is sometimes more valuable than the record of a success. Not only technologists but devotees of pure science are sometimes guilty of keeping very quiet about their failures. The result is a large amount of valuable human energy wasted in useless repetition of wearisome work. The best policy to-day is the broad-minded interchange of information, unless there are genuine and necessary trade secrets to be guarded, and these will steadily grow fewer.

Tests of Riveted Joints.

With the present requirements of large members for nearly every line of steel construction, it becomes of the utmost importance to reinvestigate the fundamental principles governing such construction before they are applied to members of much greater dimensions. The results obtained in the past by loading comparatively small members to failure, although in some cases few in number, have formed in reality the entire empirical basis of structural design. They have been of the highest value and, in general, it may be considered both rational and safe to reason from the tests of small members to the design of the largest now required, but that is not by any means the whole subject, even if the apprehensions created by the Quebec bridge failure be disregarded. The Engineering Record has already commented editorially upon the need of tests of the largest steel compression members which can be broken in any testing machine now existing, and all observations of that character applicable to compression members bear no less forcibly on riveted joints.

On the whole, a good many riveted joints which may be considered full-size have been tested to destruction, the most of such investigation having been done at the Watertown arsenal. This work, however, was performed a dozen or fifteen years ago before riveted joints had assumed anything like the magnitude in structural work which they now attain. A reference to the records of these and other similar tests will show at a glance the pressing need of an extension of that class of experimental work to joints of more nearly the proportions constantly employed in structural design of the present day. The plates then used were, in the main, much thinner than those now required, and the prevailing diameter of rivets now employed is seldom found in the older tests. Any one who has had occasion to test plates and shapes of light section will be impressed by the need of tests of joints built of much heavier plates than those of the old investigations and with the further need of using not only larger rivets to correspond with those now generally used in heavy work, but also greater numbers in the joints to be investigated. With joints of one or two rows of rivets, whether that joint be butt, lap or any other type, the distribution of the shearing stresses on the rivets or the tensile stresses in the net sections of the plates must be at least approximately uniform, eliminating any strong probability that the maximum intensity of either kind will be substantially different from the mean. That condition no longer holds, however, with three or four or more rows of rivets, which may easily be required in a structural tension joint, to say nothing of the excessive grouping of rivets found in compression joints, and allowing for the fact that the latter are usually supposedly of such a character as to permit the machined ends of plates and shapes to abut. Finally, the thicker plates produce a proportionately greater bending of the rivets. This latter is a most important consideration, for its influence in reducing the ultimate resistance of the joint is too complicated to permit its value to be computed with any sensible degree of accuracy.

The thinner plates not only leave the rolls at a sensibly lower temperature than thick plates, but they also generally have more work put upon them between the ingot and the final pass between the rolls, both of these effects tending to increase greatly the elastic limit and to some extent the ultimate resistance. A similar general observation, although of less force, may be applied to the smaller rivet in comparison with the larger. These and other material considerations go to show conclusively that while the values determined by the older tests of joints may not be, and probably are not, dangerously in error when applied to the heaviest connections now in use, there is sufficient reason to believe that either new values should be determined or the older confirmed by sufficient range of experimental work to include the full-size types now generally employed.

Public interest, at least that portion of it with industrial or technical affiliations, has been much stimulated during the past year or two by agitations designed to secure federal appropriations or funds from other quarters to be expended in full-size tests of structural members. No work of this character can be considered satisfactorily comprehensive without including the breaking of full-size riveted joints with such complete records of attendant phenomena as to solve all questions relating to the intensities and distribution of the tensile, shearing and bearing stresses developed in the various elements of the joints.

It is of special importance, among other things, to determine the effect of such unbalanced riveted connections as that of a group of rivets through the long leg of an unequal legged angle both with and without connecting lugs, and what may be taken as the real net section in such cases and others where the group of joint rivets is symmetrical about the centre line of the member, but with the outer rows of rivets diminishing in number nearly or quite to a point. In the case of the tension flange angles of a plate girder, when the rivets are staggered so as to make the net section the greatest practicable, no designer can be certain as to the value of that net section. Practice is by no means uniform in regard to these various questions of design of riveted connections, although they are of great importance as constituting the main features of the particular constructions in question. It is not possible to solve these questions of unsymmetrical arrangements of rivets or of real net sections except by full-size tests with the dimensions corresponding closely to those of actual work.

As already indicated, the apparent shear of rivets in joints, as determined by test would naturally be expected to be sensibly different from the ultimate shearing stress as determined for pure shear. The rivet has been upset to fill its hole in the process of heading, but, what is of more consequence, it is usually subjected to much bending unless the plate is thin. There are at

the present time not enough data to determine what may properly be taken as the working shear, with accuracy, in this branch of structural design. It is known well enough that when the rivet is subjected to heavy bending, its resistance to apparent shear is materially less than when such bending is absent, but a suitable allowance for thick plates has not yet been determined. Nor again can any engineer be confident of the value of rivets which bind three or four or more cover plates to the flange angles of a plate girder, supposing that the rivets fully fill their holes. Although riveted joints are constantly designed as if every element of any given case were completely known, few parts of a steel structure are in greater need of thorough experimental investigation.

Notes and Comments.

PASSENGER RATES between Pittsburg and Cincinnati on the boats of the Pittsburg & Cincinnati Packet Co. have been reduced to one cent a mile as a result of the two-cent fare on railroads required by recent legislation in Pennsylvania, Ohio and West Virginia. The loss of passenger business by the boats since the railroads reduced their fare was stated to be the cause of this cut in the rates. This does not look as if steamboats had such a tremendous influence as competitors of railways for passenger business as some inland navigation enthusiasts would have us believe.

THE CONNECTICUT RIVER BRIDGE at Hartford has been put in service, although it will not be formally dedicated until next spring. The design and construction of this imposing structure have been fully described in a number of articles in this journal, and it is therefore necessary to mention here only that it ranks among the finest masonry bridges of the world, which is only natural in view of the fact that Messrs. Alfred Boller and E. M. Wheelwright co-operated as consulting engineer and architect respectively in its design and Mr. Edwin D. Graves put so much of his ability and energy into his work as chief engineer that his health gave way under the strain about a year ago. The bridge has nine spans of lengths varying from 68 to 119 ft., its total length being 1,192½ ft. It is constructed of granite, and the simple grace of the spans and the strength of the piers make it a masonry structure well deserving a visit from those interested in bridge building.

THE ENGINEERS' CLUB of Philadelphia has recently shown good proof of its strong position by the purchase of a handsome house on Spruce St., which will be fitted up for a clubhouse. It is a four-story building, occupying a 25x170-ft. lot. Alterations will be made in it so that the second floor can be used as an assembly room for general meetings, and the first floor will have dining, smoking and meeting rooms. There will be a billiard room and other conveniences of a social nature in the building, and it is expected that with the greatly improved facilities the social side of the club will take an added importance. Attention is particularly called to this new home of the engineers of Philadelphia because it is an excellent example of what this journal considers the best thing for the engineering profession in any city. Engineering has so many branches, and those following them so rarely come together in a business way, that a club of this sort, combining both social and professional features, is highly desirable in any city large enough to have many engineers. What is wanted at the present time more than anything else in engineering circles is some centripetal tendency which will draw together all those who are engaged in designing or constructing works and machinery in any district. Until engineers get together socially and learn to know and respect each other personally there will be lacking that bond of good fellowship which has enabled physicians and lawyers to act unitedly whenever their interests required it. The Engineers' Club of Philadelphia is doing just this kind of good work, and its success should lead local engineering organizations elsewhere to strengthen their position in the same manner.

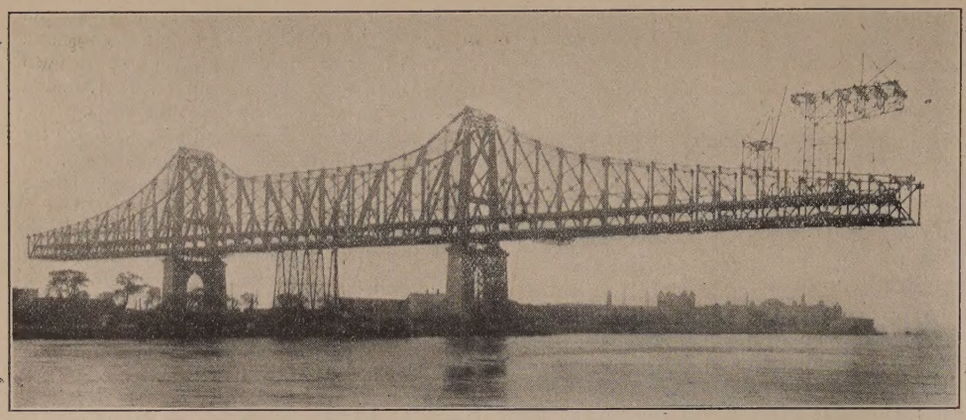
THE LIGHT MOTOR exhibited at the Paris Automobile Salon by M. Robert Esnault-Pelterie is interesting as bringing a little closer the practicability of aerial navigation. Few people realize that the distance between impossibility and possibility is almost reached, and that the engineer is the one who has the task of reaching and passing it. The successful construction of powerful motors of light weight, which has made the automobile a standard product, is an indication of what may be expected when more time and study have been spent on motors for aviation. The motor referred to weighs 2.86 lb. per horsepower, but its accessories and the propellor bring the total weight up to about 5.5 lb., and it is considered by some engineers who have looked into the subject that the greatest promise of successful reduction in weight is offered by a redesign of accessories and possibly by the use of some other source of power than gasoline. It is difficult to see how much more weight can be eliminated from the Esmault-Pelterie motor.

SOCIETY AFFAIRS are managed in some respects differently in Great Britain from the way they are here, as is evident from an announcement in the "Architect," of Dec. 6. It is given here in all its simplicity: "The Council of the Royal Institute of British Architects are prepared to receive applications for the appointment of secretary. Applicants, who need not necessarily be architects, should possess some literary capacity, a thorough knowledge of French and a working acquaintance with other languages. Age not less than about thirty or more than about forty-five years. The secretary will be required to devote his whole time to his official duties. Salary £500 per annum. Any canvassing will disqualify. Particulars may be obtained by writing to the Honorary Secretary, 9 Conduit St., W., not later than the 21st inst., and applications must be received by Dec. 31." It is worth while considering whether such a method of filling a vacant secretaryship of a great national professional organization may not have advantages when American societies have to appoint such officers, provided the constitutional requirements permit the procedure.

CENTRAL PARK, New York, is one of the city's most precious possessions. Visited during most of the days of the week it seems given over to wealthy drivers, riders and motorists, but on Saturday afternoon and Sunday it amply repays all the money spent on it. Children who otherwise would never know of birds and flowers, tired and worn men and women whose conditions of life have little enough sunshine and happiness to offset stern struggles, young people who see nothing attractive in Coney Island dance halls and the back rooms of saloons, fill the Park on such days. In the summer, particularly, this great open space for the people is a self-evident blessing to the city. It is therefore surprising that the municipal fathers pay no attention to the fact that the Park is wearing out. Five years ago, Mr. Samuel Parsons, who is the technical director of the Park Board's work, gave notice that the vegetation was dying out, but he was merely ridiculed then. Now anybody who knows about such things can see this for himself. Many of the trees and shrubs are past maturity and should be replaced. The thin layer of soil on the rocky bed of much of the Park needs mould and fertilizer, and a great amount of sodding must be done. The drives should be reconstructed and, as a matter of public health as well as public pleasure, the drainage of the land must be remodeled. These and other things will require about two and a half million dollars. It is a large sum, but it can be furnished in installments during a period of five years. It must be furnished before long or this chief pride of the city will deteriorate rapidly, and a place that has meant health as well as happiness to thousands of people of little means will lose much of its charm.

THE SETTING OF CEMENT has been discussed a great many times but it is safe to say that very rarely has a paper on the subject contained so much material for careful consideration as that by Messrs. Henry S. Spackman and Robert W. Lesley, printed in this issue. Mr. Lesley is not only a pioneer manufacturer of Portland cement in this country but he is also a student of the technology of the industry with which he is prominently identified. He is a firm believer in the obligation a successful manufacturer owes to the welfare of his industry, a belief that led him to endow the Lesley cement laboratory of the University of Pennsylvania. It has further led him to plan investigations into various aspects of cement and concrete that are still matters of dispute, and, with the co-operation of the Spackman Engineering Co., to carry on some most interesting research work, begun several years ago. The first fruits of this work was a paper on ancient Roman mortars read before the Association of American Portland Cement Manufacturers some time ago, and the second result is the paper printed on page 691. More studies are being made, and the results of all this work will form one of the most important contributions to our knowledge of structural materials ever made by an American manufacturer.

THE GRADE CROSSING PROBLEM was discussed very plainly by President Ralph Peters of the Long Island R. R. recently. He pointed out that notwithstanding the efforts of railroad companies to reduce the number of such crossings and the laws in many States providing for a joint expenditure of railroad and public funds for their elimination, there is a constant and persistent demand in every community for constructing more of them. The development of land schemes, of additions to towns and cities, the natural growth of communities in sections adjacent to railroads, have created new lines of travel and resulted in opening new streets and establishing new grade crossings. These crossings, when rightfully opposed by railroad companies, have been the cause of prejudice against corporations endeavoring to protect public interests as well as their own. The railroad is not so much responsible for the grade crossings as the public, for railroad managers are more than anxious to prevent more of them and to eliminate those in existence. They are ready to join in doing away with them, but feel that the work must be done with the co-operation of the communities and, in many cases, at joint expense. Mr. Peters urges that until all grade crossings have been eliminated, the public should not forget its joint responsibility for their existence and should use such crossings with great care. This opinion of the head of the Long Island R. R. is substantially that expressed in the laws of Massachusetts, the State where the respective rights of the public and of public-service corporations have received the most thorough study and the most equitable legislative treatment.



Appearance of Bridge on June 15th of this Year.

THE ERECTION OF THE ANCHOR ARMS OF THE BLACKWELL'S ISLAND BRIDGE.

The 630-ft. island span and the adjacent 591-ft. and 492-ft. cantilever arms of the 1,182-ft. west channel span and the 984-ft. east channel span have now been virtually completed, making a continuous structure 1,713 ft. long. This portion of the bridge has a total weight of about 48,700,000 lb. and is 135 ft. high above low water and about 320 ft. high to the tops of the main bents. Riveting, the construction of the upper floor, and other minor operations are now in progress on the cantilever arms, but the main travelers and steel falsework have been removed and are now in service for the erection of the anchor arms, 469½ ft. and 459 ft. long, on the Manhattan and Queens shores respectively.

Underneath each anchor arm a large storage yard has been enclosed by a high fence and in it a full length 85-ft. gauge track has been laid on which there is one of the two 65-ton gantries originally provided for the Blackwell's Island storage yard. Timber docks have been built near the piers at the water's edge, on both sides of the river, and equipped with 65-ton steel derricks with which the riveted members are unloaded from car floats and transferred on service tracks to the gantries, which pile and store them and finally deliver them to other service cars carrying them under the anchor arms within reach of the erection travelers, as they are needed. Shipments were commenced in January last, and since then about 11,000 tons of steel have been received at the Manhattan end and about 7,000 tons of it have been erected there. At the Queens end, about 8,000 tons of steel have been received since March, and about 4,700 tons were erected up to Oct. 1. The construction of steel falsework for the Manhattan anchor arm was commenced in March, and erection was commenced in June and has since been carried on with a force of 75 to 150 men. For the Queens anchor arm the erection of falsework was commenced in April and of steel in July, and about 1,700 tons of steel were erected up to Oct. 1, with a corresponding number of men. The total weight of steel erected on Oct. 1 was about 68,000,000 lb.

The erection of the Manhattan anchor arm was commenced by a stiff-leg wooden derrick with a 50-ft. 10-ton boom seated on top of pier 1, which built up the first two steel falsework towers and assembled on top of them the falsework traveler. This has a 33x57-ft. platform built up of stringers and floor-beams borrowed from the permanent structure, and carries one of the standard steel derricks with a 75-ft. boom of 65 tons capacity.

After several panels of the steel falsework had been erected and the falsework traveler advanced some distance beyond pier 1, the wooden derrick there commenced the erection of the No. 1 derrick used to assemble the permanent trusses. This traveler is of very simple construction, with a steel girder platform carrying two stiff-leg steel derricks with their masts braced together horizontally and diagonally, and having 65-ton booms, 85 ft. long. The traveler is operated by two special Lidgerwood engines and weighs about 300 tons with its rigging.

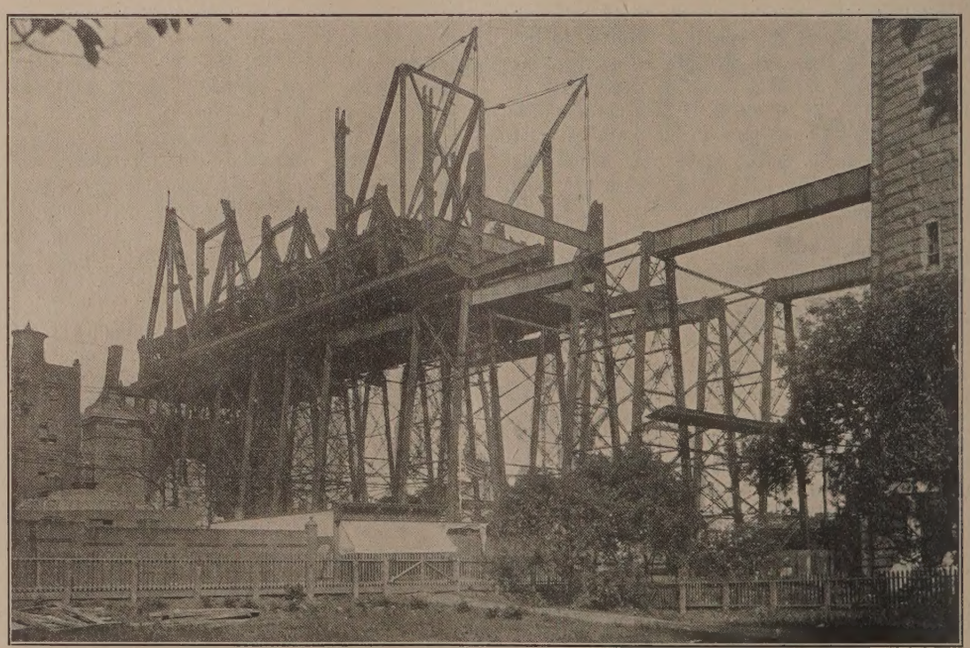
On the Queens anchor arm, traveler No. 1 has already progressed from pier No. 1 almost to the anchor pier, and has nearly completed the erection of the span up to the middle of the truss. On the Manhattan anchor arm, traveler No. 1 has now reached the anchor pier and completed the erection of the lower half of the truss, and the erection of the upper half has been commenced by traveler No. 2. This traveler has a Z-shaped side elevation and consists of a 36x40-ft. steel tower 124 ft. high, with an extended base and 63-ft. top overhang on the forward side to provide for the erection of panels of the truss in advance of the traveler for the cantilever arm. Although this function of the traveler is not necessary for the anchor arm erection, the large number of tackles are still suspended from it and it is equally serviceable in building the upper part of these trusses. This traveler, like No. 1, is operated by two special steam hoisting engines and, with its rigging and other equipment, weighs about 550 tons and requires a force of 60 men for its operation. No 2 traveler is not yet entirely completed for the Queens anchor arm, but has been finished and in service for some time on the Manhattan arm, where its operations were at first supplemented by traveler No. 1, which for a time worked simultaneously with it erecting the upper parts of the trusses in the panels next the anchor pier as the traveler returned from that point toward the river.

The removal of the No. 2 travelers from the cantilever arms over the main channels was a work of some difficulty and was accomplished by means of the stiff-leg derricks seated on top of the travelers for use in erection operations and by a derrick and special falsework tower alongside. The first mentioned derrick was used for the removal of the overhang and part of the upper portion of the traveler. The remainder of the traveler was handled by a 10-ton 50-ft. derrick on top of a 29½x57-ft tower, 24 ft. high, seated adjacent to the traveler on the top lateral struts of the bridge. The tower was made with two transverse X-braced panels and a center post to carry the foot of the mast, which was supported on a small auxiliary tower, with bearing on the upper deck of the bridge.

Derrick No. 2 was re-erected on the Manhattan anchor arm, with a 13x18-ft. wooden tower, 70 ft. high, set at panel point 15. This tower was simply used as a support for the mast of the wooden stiff-leg derrick, with a 70-ft. boom of 10 tons capacity. The feet of the stiff-legs were anchored to the permanent trusses at panel point 16.

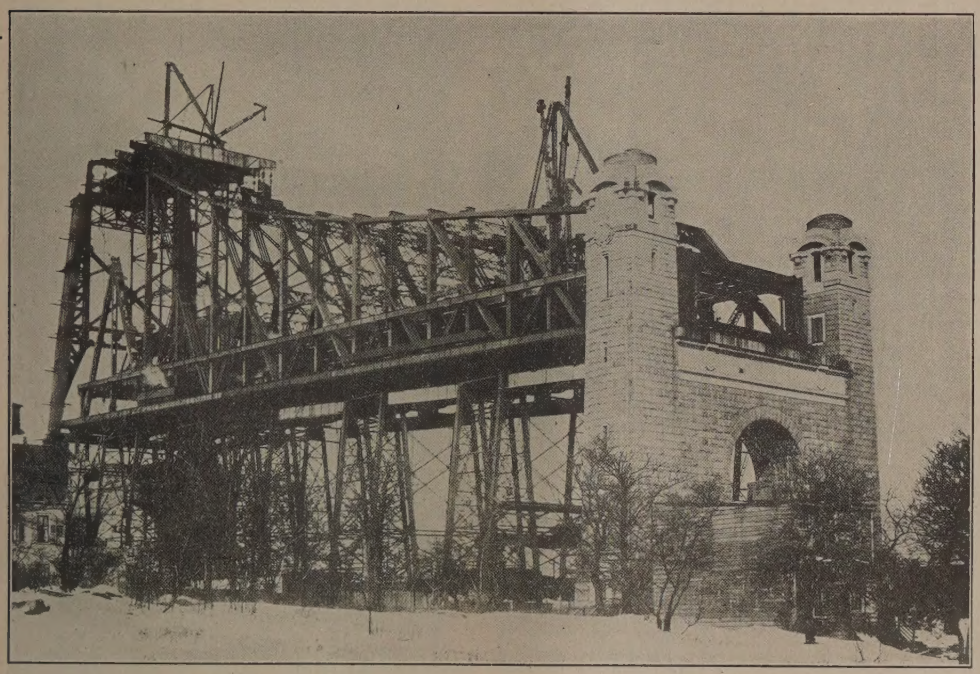
The booms for the large steel derricks were originally 75 ft. long, with a capacity of 65 tons. Since the completion of the island span and river cantilever arms they have been extended to a length of 85 ft. and reinforced to handle 75 tons. The rectangular cross section is now made up of two built channels, each with a 29x½-in. web plate and two 6x4-in. flange angles, latticed and connected by diaphragms 7½ ft. apart on centers. The links for connection with the topping lift tackle are six 6x¾-in. eye-bars 6 ft. 4 in. long, two of the center ones being bent to secure the proper spacing on the 3 and 5 in. pins. The bars are fixed in their relative positions with tie bolts and gas pipe separators. Similar links are provided for the hoisting tackle.

The plans and operations for the erection of the anchor arms correspond almost exactly with those for the island span and the cantilever arms,



Erection of Lower Part of Queens Anchor Arm with No. 1 Traveler.

described in previous issues of this journal. All of the principal web members of the trusses are erected in halves, spliced near the center of the height of the truss, the heaviest member being the 80-ton semi-diagonal nearest the main pier. As fast as the truss members are put in position temporary connections are made by inserting wooden pins 1 or 2 in. smaller in diameter than the holes, and later following them with the permanent pins preceded by ordinary sectional pilot nuts. The eye-bar members are assembled in complete panel groups in the storage yard, where they are accurately spaced and strongly clamped with steel and wooden yokes, after which they are handled like compression members as single units, the largest containing twenty separate bars and weighing about 65 tons. All truss pins are driven with an ordinary steel ram weighing about 2 tons. This has proved thoroughly efficient, so that it has not been necessary to use the hydro-pneumatic ram provided for this purpose. The 185-ft. vertical bents on piers 2 and 3, which weigh about 1,200,000 lb. each and were erected 8 in. out of plumb at the top, so as to provide for the reverse camber in the island spans, have gradually approached their final positions and are now exactly vertical and correspond to their calculated dimensions and locations.



Erection of Upper Part of Queens Anchor Arm with No. 2 Traveler.

There are in all about 752,000 field-driven rivets in the bridge, of which 125,300 are 1 in. in diameter, 426,850 are ⅞ in. and 199,860 are ¾ in. Of these about 37,500 in the island span and adjacent cantilever arms are all driven. Field riveting on the Manhattan anchor arm is in progress in most of the vertical bents and includes pracically all of the rivets which can be driven without interfering with the possible adjustment of the members to take their share of the final stresses. All chord splices in the anchor arms are left entirely undriven, and no rivets are driven in the stringers at the main panel points. One end of each truss diagonal is left unriveted, and one end of each lateral diagonal or sway-brace member is also left unriveted.

On the island cantilevers a very large number of rivets in the bottom lateral system, floorbeams, stringers and lower chord splices were driven from a movable platform suspended underneath the lower deck of the bridge, which greatly facilitated the work and diminished the danger. The platform is 98 ft. long, transverse to the bridge axis and 26 ft. wide, and has a solid wooden floor protected by a hand rail and supported on a pair of wooden lattice girders 9 ft. deep and 98 ft. long. Long steel connection plates secured to the top chords of the girders are provided with a number of bolt holes to receive the lower ends of the suspenders. The latter are stiff vertical members made of angles and having hooked upper ends engaging the bulb angles which serve as guards on the outer flanges of two of the track stringers.

The platform was originally designed to have roller bearings on these angles, but after construction it was found easy to move it forward without the rollers, simply sliding on the angles by means of two small tackles anchored in advance and operated by hand. The roller bearings were therefore dispensed with, and the platform is rapidly moved back and forth by the men on it. Double sets of suspenders are provided, and when one set interferes with the floorbeams the other set is connected to the platform on the opposite side of the floorbeams, and afterwards the rear set is removed and the platform advanced. This arrangement saves time and expense otherwise necessary for placing numerous small riveting scaffolds and affords much greater convenience and safety for the riveters and painters.

On the lower deck of each of the anchor arm spans there is a small building, which contains an Ingersoll-Rand compressor of a capacity of 200 cu. ft. of free air per minute, driven by a Westinghouse 50-h.-p. electric motor. A special feature of the arrangement is the application to the motor of a pneumatic controller connected with the air in the receiver and adjusted to shut off the current in the motor when the pressure reaches 105 lb. per square inch and to turn it on when the pressure falls to 95 lb. The governor is of the standard type made by the Cutler-Hammer Manufacturing Co. The cooling water from the compressor is circulated through a Koven atmospheric cooler, where it falls in a thin sheet successively over a large number of horizontal steel discs arranged in a verical line, so as to cause a number of cascades. About thirty-two gangs of riveters are kept constantly at work, and the fitting-up for them is accomplished by four Philadelphia pneumatic reamers, which are also able to drill all the new holes required and frequently drill out bolts, rivets or drift pins that have to be removed. Compressed air at about 100 lb. is distributed parallel to the axis of the bridge through a screwed steel pipe 3 in. in diameter at the receiver, which gradually reduces and is provided with numerous valved outlets at convenient intervals.

There are about thirty pneumatic hammers used for general riveting, but for the rivets in the web splices of the bottom chords special pneumatic machines have been provided. These are designed to work in the 15-in. space between the webs, which could hardly be reached by ordinary methods. One machine has an X-shape framework, with two curved arms about 10 ft. long, intersecting on a common pivot like a pair of shear blades. The extremity of one long arm carries a pneumatic hammer, opposed to which is a holding-on die attached to the extremity of the other long arm. A pneumatic cylinder is placed at the opposite end of the machine, between the extremities of the short arms, to both of which it is connected. The long arms are set on opposite sides of the webs in which rivets are to be driven. The die is placed in engagement with the rivet head, and the pneumatic cylinder operated to separate the short arms and correspondingly converge the long arms, bringing the pneumatic hammer down close on the point of the rivet which is being held by the cap. The hammer is put in operation in the ordinary way and drives the rivets. Four of these machines are in service and are doing excellent work, having a capacity from 200 to 400 1-in. rivets daily.

Two other special riveting machines are so small and compact that they can be placed entirely within the chord in the 15-in. space between its webs. The cast-iron housing contains three horizontal pneumatic cylinders, transverse to the bridge axis. The center one, 4 in. in diameter, operates an ordinary pneumatic riveting hammer at one end of the machine, while the other two, 2 in. in diameter, operate cylinders at the opposite end on both sides of the machine, which, when pressure is admitted, are forced against one web of the chord and push the machine and pneumatic hammer against the opposite web, where the hammer engages the rivet to be driven. The latter is maintained in position by an ordinary pneumatic holder-on outside of the chords. This machine drives about the same number of rivets as the one previously mentioned, and both of them require four men to operate them. The rivets driven by them are 1 in. in diameter and about 6 in. grip. The painters follow the riveters and use the same suspended platforms.

The bridge was designed and its construction was executed under the direction of the Department of Bridges, of which Mr. J. W. Stevenson is now commissioner; Mr. C. M. Ingersoll, Jr., chief engineer; Messrs. O. F. Nichols and Albert Lord Bowman, consulting engineers; Mr. J. D. Wilkins, assistant engineer in charge of design, and Mr. J. B. Knighton, resident engineer. The Pennsylvania Steel Co. is the contractor for the fabrication and erection of the steel work.

AN ELECTRIC RAILWAY of considerable technical interest is operated by the Indianapolis & Louisville Traction Co. between Seymour and Sellersburg, Ind., a distance of 41 miles. The power-house is located at about the center of the route and direct current is transmitted from it at 1,200 volts, measured at the station between the single OOOO grooved trolley wire and the rails. There is an entire absence of high-tension feeder wiring and sub-stations. The station has two 26x48-in. Allis-Chalmers engines operating non-condensing at 120 r. p. m. These drive in pairs four 300-kw., 600-volt General Electric generators with their armatures mounted commutator to commutator on the engine shaft.

Retaining Walls on the Delaware, Lackawanna & Western R. R. at Buffalo.

The Delaware, Lackawanna & Western R. R. has just completed the reinforced concrete retaining walls for its track elevation work in the southeastern part of Buffalo, N. Y. Two general types of walls have been used, one for the sections below 12 ft. in height, and the other for sections from 12 to 24 ft., the last height being that of the maximum section. A buttressed wall is used for the higher sections, but because the saving effected by this type would not offset the extra cost of forming for the buttresses a plain L-section without buttresses was adopted for walls under 12 ft. in height. The forms for the latter are no more expensive than would be required for a plain concrete wall, while the saving of the L-section in concrete is evidently quite a material one. A wall with a toe projecting beyond the front face and under the sidewalk was first contemplated but objections by the authorities of the city of Buffalo prevented the adoption of such a design.

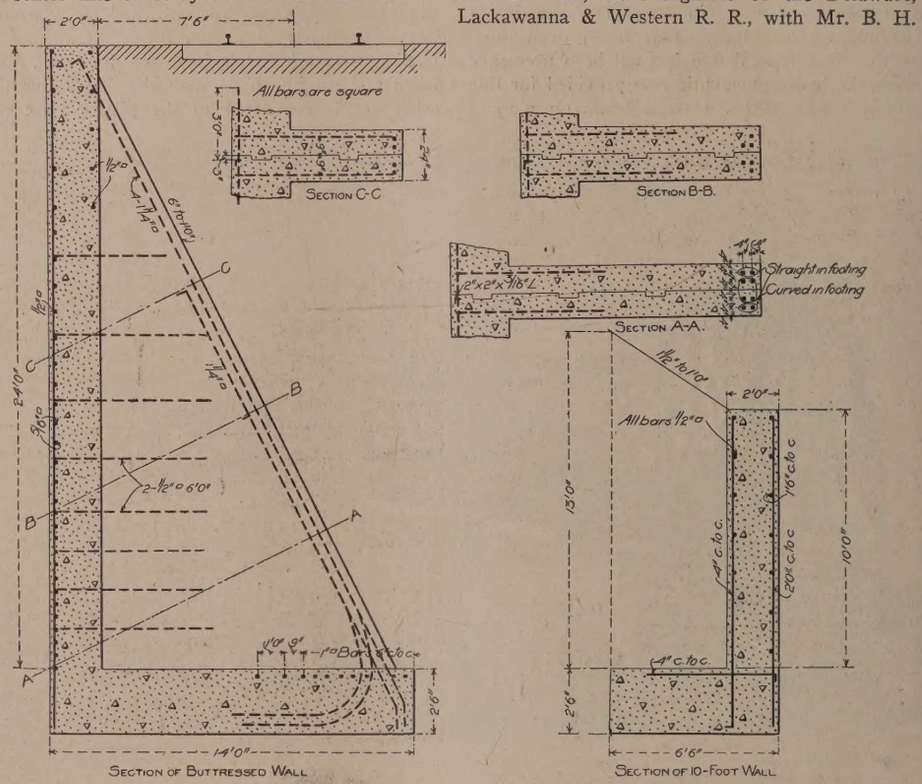
The tracks were placed near retaining walls only where the buttressed section was used, the distance from the center of the track to the back of the wall in no case being less than 7 ft. 6 in. No tracks whatever were close to the section without buttresses, the retained fill having a long slope up to the roadbed level. The earth pressure against the walls, which are 2 ft. thick, was figured according to the formulas in Church's Mechanics, the weight of earth being taken at 100 lb. per cubic foot. The weight of concrete was taken as 150 lb. The pressures due to the moving loads on the tracks were figured for Cooper's E 50 loading. The width of the base for the buttressed type was taken as half the height of the wall above the top of the base plus 2 ft., and for the L-section as half the distance from the top of the base to a point where a vertical line through the inner edge of the base intersects the slope line, the slope being 1½:1. The base is 2 ft. 6 in. thick. The stresses in the steel were kept under 16,000 lb. per square inch and in the concrete under 500 lb.

The reinforcement of the buttressed section consists of both horizontal and vertical bars near the face of the wall, horizontal bars in the base and inclined bars in the buttresses, square mild steel being used in all cases. Those laid horizontally in the face of the wall are of ⅝-in. material spaced on different centers, farther apart at the top and closer at the bottom, to take care of the increasing pressures toward the bottom of the wall. The vertical bars in the front face are of ½-in. material spaced throughout on 2-ft. centers. In the back face five ½-in. bars have been placed on 18-in. centers. Tie rods run from the wall into each buttress, two of them being looped over every third of the horizontal face bars. They are 6 ft. long and ½ in. square. The heaviest bars used on the work, 1¼ in. square, are used for the diagonals in the buttresses. There are eight of these, but only four run the entire distance from the top of the wall to the outer end of the base, the others starting at intermediate points. At the bottom of the buttresses, four of the diagonal bars are bent back toward the wall and are embedded near the under side of the base. Running longitudinally in the top of the latter are 1-in. square bars, in the outer half of the footings. The rods in the face do not come closer than 2½ in. to the surface of the concrete, and those in the buttresses and in the base not closer than 3½ in.

In order to preserve the proper spacing for the horizontal bars in the face a 2 x 2 x 3-16-in. angle is placed in the face at each buttress, holes being punched in one leg to hold the bars in their proper positions. They are left in place and form part of the permanent reinforcement of the wall. The horizontal bars are on 18-in. centers for the first 6 ft. down from the top, on 12-in. centers for the next 6 ft., on 9-in. centers for the next 5 ft. 3 in., and on 6-in. centers for the remainder of the distance to the bottom of the wall.

The reinforcement for the sections below 24 ft. and above 12 ft. in height is substantially the same for all sections, except that the base would be moved upward cutting off the lower part of the reinforcement shown in the diagram, so that from the top down for any given distance the steel will be the same in walls of all heights.

The buttresses are 2 ft. wide and 12 ft. 6 in. on centers. In the original design it was intended to place an expansion joint every 25 ft., or on the center line of every second buttress. When construction was started, however, it was found that the wall could be more economically placed in 50-ft. than in 25-ft. sections and it was therefore decided to place the expansion joints every 50 ft., so that the day's work could be made to terminate at an expansion joint.



Reinforcement in Buttressed and L-Sections.

All of the horizontal bars are 28 ft. long, while the others, necessarily, are cut to shorter lengths in accordance with their positions. The concrete was a 1:3:6 mixture made with gravel.

For the lower walls without buttresses the reinforcement is similar to that of the 10-ft. wall in the diagram. For this particular height it consists of ½-in. square bars in the base and in both faces of the wall. The vertical bars in the front face are on 2-ft. centers, and the horizontal bars on 18-in. centers, while in the back face the vertical bars are on 4-in. centers and the horizontal ones on 18-in. centers. The bars in the base are placed horizontally on 4-in. centers and run from the front of the wall back into the footing.

An accompanying photograph shows clearly the placing of part of the reinforcement before the concrete was cast. As will be noted, great care was exercised in holding the steel in its proper place. A board notched on both sides with the proper spacing for the diagonal bars was used to prevent the displacement of the steel. Similar notched boards were placed in the base to hold the bars in proper position, and at the upper ends of the diagonal bars in the buttresses. For the latter purpose the notched piece was nailed to a bracket attached to the front form for the wall. The vertical and horizontal bars, in addition to these precautions, were wired at intersections so as to insure against displacement. The concrete was placed very wet and on this account long ½-in. rods threaded at both ends were used to tie the forms together.

The walls are now completed and although on account of the soft nature of the ground along the right of way, all of them rest upon piles, no cracks have occurred. Of the maximum section, 24 ft. high, there is a total length of 1,700 ft., and of this length 1,000 ft. are in one continuous wall.

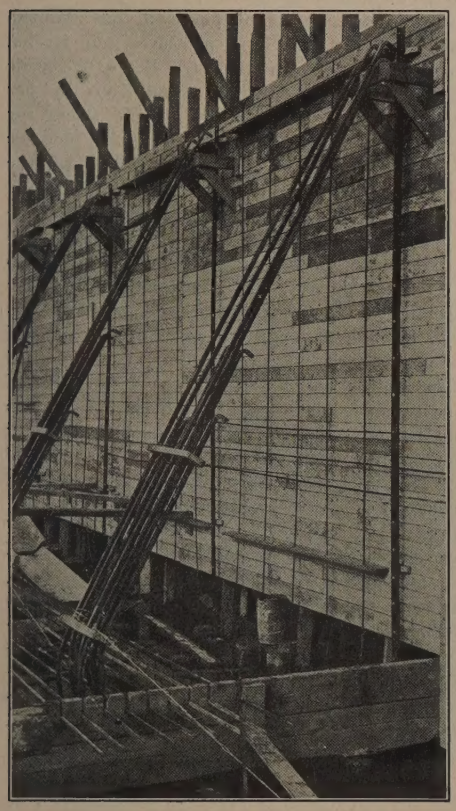
The wall was designed under the direction of Mr. L. Bush, chief engineer of the Delaware, Lackawanna & Western R. R., with Mr. B. H. Davis in immediate charge of the work. The field construction was in charge of Mr. R. M. White, resident engineer.

Competitive Designs for the proposed municipal office building for Greater New York have been invited by the city from thirteen architectural firms. The building, which is to cost about $8,000,000, is to stand near the Manhattan end of the Brooklyn Bridge and its lower floors will be used as the Bridge terminal for surface, subway loop and elevated trains. The floor space for offices, exclusive of halls, shafts and courts, is 600,000 sq. ft., necessitating a building 20 stories high, which will accommodate all but two of the municipal departments. The firms invited to compete are J. Stewart Barney, Carrère & Hastings, Clinton & Russell, J. H. Freedlander, Cass Gilbert, Heins & La Farge, Helmle & Huberty, Hoppin & Koen, Howells & Stokes, H. R. Marshall, McKim, Mead & White, Trowbridge & Livingston and Warren & Wetmore. It is understood that Mr. Cass Gilbert will not compete. Each firm will receive $1,000, and the winner will receive $5,000 for his design if the city does not accept it. The judges agreed upon are Frank Miles Day, president, American Institute of Architects; William A. Boring, vice-president, American Institute of Architects, and Francis C. Kimball, designer of many large New York office buildings. The designs are to be submitted April 15, and the decision announced on May 14, 1908.

Pipe Lines for Hydraulic Power Plants.

The technical staff of the Telluride Power Co. have organized the Telluride Institute, and at its meetings papers of an engineering character are presented for discussion. One of these, by Mr. Arthur Jobson, was on pipe lines for hydraulic power plants, and an abstract of it follows:

The determination of the most economical diameter for the penstocks of high-head hydraulic power plants is often a troublesome problem to the engineer. The principal factors on which the solution of this question depends are quantity of flow, static head, probable total cost of development per horse-power, and cost per pound of the pipe or penstock erected. Its proper solution is particularly important should the proposed plant be one of high head, in which case the cost of the penstock will be a large percentage of the total cost of the installation.



Methods of Holding Reinforcement in Place.

The power lost in a pipe line per unit of length for a fixed quantity of flow varies inversely as the fifth power of its diameter. On the other hand, its cost per unit of length for a fixed static head will vary as the square of the diameter. Hence it follows that should a pipe of very large diameter be selected for the purpose of increasing the efficiency of the pipe line, the value of the extra amount of power obtained due to this higher efficiency may not merit the increased investment required. It also follows that should a very small size be selected for the purpose of reducing the annual charges on the investment, the resulting loss due to the lower efficiency of the pipe line may more than offset this reduction in annual cost. As a result it is important in fixing the diameter of penstocks to consider both the cost of the pipe installed and the loss of power in the pipe during the operation of the plant. These losses should be balanced, one against the other, in such a manner that the resultant due to each factor acting in coincidence, shall be as small as possible, consistent with the total first cost of the plant and its operating expenses.

In deriving an expression for the most economical diameter of penstocks, two methods of procedure may be adopted: (1) The yearly interest and depreciation on the first cost of the pipe may be obtained in terms of its diameter, and also the value of the power lost in the pipe line per year in terms of the diameter. Evidently the sum of these two expressions will be the total loss per year due to each factor acting in coincidence, and if this sum is derived with respect to the diameter, a diameter will be obtained such that the sum of these losses will be a minimum. (2) The sum of two ratios may be obtained in terms of the diameter; one ratio being that of the total cost per unit of length of the pipe installed to the total cost of the proposed installation, and the other, the ratio of the power lost in the pipe to the total rated capacity of the plant. If this sum be derived with respect to the diameter of the pipe, a diameter will be obtained such that the sum of these ratios will be a minimum.

Either one of these two methods will result, theoretically, in the most economical size of pipe. In developing this expression the second method will be adopted, the values of the letters and symbols used being represented as follows: Q = quantity of flow in cubic feet per second; V = velocity of water in feet per second; D = diameter of pipe in feet; t = thickness of pipe in feet; S = safe tensile stress of metal in pounds per square foot; P = internal pipe pressure in pounds per square foot; A = internal cross-sectional area of pipe in square feet; h = static head in feet; h' = loss of head in feet due to pipe friction; f = coefficient of pipe friction; c = cost in dollars per pound of installed pipe; k = weight of metal used in pounds per square foot; N = total cost of proposed installation in dollars; C = cost of pipe installed per unit length; H = total cost of installation per horse-power.

The following well-known formula represents the loss of head per unit of length in a circular pipe flowing full of water:

$$h' = 2\,f\,V^2 \div Dg \qquad (1)$$

Since $V = Q \div A = 4Q \div \pi D^2$, by substituting this value for V in (1), $h' = 32\,f\,Q^2 \div \pi^2\,g\,D^5$. Assuming the combined efficiency of a water wheel and generator to be 75 per cent. for large units, the following is the expression for the horse-power developed from a water-power installation: Horse-power $= 0.0851\,Q\,h''$, where h'' is the effective head. Hence the loss of power per unit length of pipe due to friction will be given by:

$$\text{Horse-power lost} = 0.0851\,Q\,h' \qquad (3).$$

Substituting the value already derived for h' in (3):

$$\text{Horse-power lost} = 2.723\,f\,Q^3 \div \pi^2\,g\,D^5 \qquad (4).$$

Let $H.\,P.$ equal total horse-power of plant, and r' equal ratio of horse-power lost to $H.\,P.$ Then,

$$r' = 2.723\,f\,Q^3 \div \pi^2\,g\,D^5\,H.\,P. \qquad (5)$$

The volume of the pipe per unit of length will be given by $t\,\pi\,D$, and its weight per unit of length by $k\,t\,\pi\,D$. Hence,

$$C = c\,k\,\pi\,D\,t \qquad (6).$$

But, $t = P\,D \div 2\,S$, and, $P = 62.4\,h$. Therefore, $t = 31.2\,h\,D \div S$. Substituting this value for t in (6), gives,

$$C = 31.2\,c\,k\,\pi\,D^2\,h \div S \qquad (7).$$

Let r'' = ratio of C to N.

$$r'' = 31.2\,c\,k\,\pi\,D^2\,h \div S\,N \qquad (8).$$

Hence,

$$r'' + r' = 31.2\,c\,k\,\pi\,D^2\,h \div S\,N + 2.723\,f\,Q^3 \div \pi^2\,g\,D^5\,H.\,P. \qquad (9)$$

This last expression is the one sought for, namely, the sum of the ratios of the cost of the pipe per unit length to the total cost of the installation, and the power lost per unit length of the pipe to the available power, in terms of the diameter, D. If this expression is derived with respect to D and equated to zero, the resulting value of D will be that for which the sum of these ratios will be a minimum. Let $R = r'' + r'$.

$$\frac{dR}{dD} = \frac{62.4\,c\,k\,\pi\,D\,h}{S\,N} - \frac{13.615\,f\,Q^3}{\pi^2\,g\,D^5\,H.\,P.}$$

Equating this expression to zero and solving for the power of D, we get:

$$D^7 = 0.218\,f\,S\,N\,Q^3 \div c\,k\,\pi^2\,h\,g\,H.\,P. \qquad (10).$$

Let H equal total cost of installation per horse-power.

Long Stretch of 24-Foot Retaining Wall on Lackawanna Track Elevation, Buffalo.

$D^7 = 0.218\ f\ S\ H\ Q^3 \div c\ k\ \pi^7\ h\ g$.....(11).

The safe tensile stress of ordinary steel may be taken at 8,000 to 10,000 lb. per square inch and its weight 490 lb. per cubic foot. Assuming 8,000 lb. per square inch as the value of the safe tensile stress, and its weight per cubic foot as given above; also reducing π and g to their proper numerical values,

$D = \sqrt[7]{}\ (0.5145\ f\ H\ Q^3 \div c\ h)$.....(12).

Allowing S to remain in the formula, we get:

$D = \sqrt[7]{}\ (0.0000004466\ f\ S\ H\ Q^3 \div c\ h)$..(13).

Several objections will no doubt arise as to the value of the above derived formula in actual practice. Foreseeing a few of these possible objections and questions, they will now be discussed before attempting to make an application of the formula.

The first question which may arise is: Does the value of D as given in this equation represent a fixed value for the total length of the pipe to be installed, or is it a variable quantity depending upon the static head impressed upon each unit of the pipe? It will be remembered that h is the total, or static head under which the proposed plant is to operate. It will also be seen that the cost of the pipe installed was computed on the basis of unit length, and furthermore, that in making this computation the diameter and thickness of this unit length was assumed constant. But in the final cost formula, $C = 31.2\ c\ k\ \pi\ D^2\ h \div s$, t is eliminated, its value being replaced in terms of h. Hence it follows that the cost of the pipe per unit of length is a direct function of the static head impressed upon that particular unit length, and from this it follows that the value of D as given by equation (13) is the most economical size of pipe for that unit length and the static head impressed upon it. Evidently then D is not fixed for any particular installation, but varies as the static head upon the different parts of the pipe varies. In computing the most economical size of pipe at the power station, h represents the actual or static head of the installation, but in computing the most economical size of pipe for the same installation under half the static head of the station, then the value of h to be substituted in the final formula will be represented by one-half the actual head. Hence the interpretation of h will have to be changed. Instead of defining it as the actual head of the installation in feet, it will now be defined as the static head impressed upon a unit length of pipe for which the most economical size is being computed.

No serious questions can arise regarding the values of S and Q in the final formula. The tensile strength of the metal to be used in the pipe may be determined very closely, and when once determined, the value of S then depends upon the allowable factor of safety. The quantity of flow is usually one of the first factors to be definitely fixed in a proposed hydraulic power development, as this and the effective head are the two factors which determine the rated horse-power capacity of the plant. However, some allowance should be made for load factor in fixing the value of Q, as it is the average quantity of flow which should be used to determine the proper size of pipe rather than the maximum.

The most serious questions which may arise in making a practical application of this formula are those which will be suggested in making substitutions for f and H; the coefficient of pipe friction, and the cost per horse-power of the proposed installation in dollars. Each of these quantities are functions of D. Rankine gives $f = 0.005\ (1 + 1/12\ D)$. This, however, is not strictly true. Experiments show that the value of f depends not only upon the diameter of the pipe, but also upon the velocity of flow. J. T. Fanning gives in "Hydraulic and Water Supply Engineering," p. 233, several tables showing the value of f for various pipe diameters and velocities of flow. These values were determined by experiments with smooth iron pipes, and are probably correct, within a reasonable degree of accuracy. In these tables it is shown that for a pipe 18 in. in diameter, f varies from 0.00559 to 0.00451 at velocities of flow ranging from 1 to 20 ft. per second; a decrease of approximately 20 per cent. If the velocity of flow is fixed at 20 ft. per second, then f varies from 0.00481 to 0.00381 for pipes varying in diameter from 12 in. to 36 in., a reduction again of approximately 20 per cent. In any case the value of f should easily be fixed within 10 per cent. of its true value, since for a fixed quantity of flow a variation in velocity of flow of from 1 to 20 ft. per second would result in a decrease of diameter in the ratio of 4.5 to 1. Assuming that it is possible to select the value of f within 10 per cent. of its true value for the most economical size of pipe, then the resulting error in computing the diameter of the pipe from equation (13) will be the seventh root of 10 per cent., or approximately 1.4 per cent. From this diameter of pipe which is in error 1.4 per cent., the velocity of flow may be easily determined, which will be in error approximately 2 per cent. But for these slight errors in both D and the velocity of flow, the resultant error in the selection of f from the tables will be almost inappreciable, and in substituting this new value for f in equation (13) for recomputing D, evidently this error will be inappreciable in the diameter since the seventh root of the error will be taken. Hence one computation of D, with an assumed value of f, is all that is necessary to determine the true value of f within reasonable limits.

As has already been mentioned, the cost of a hydraulic power plant is a function of the high-pressure pipe used in the installation. Also, in the substitution for the value of H in equation (13), the objection may be made that the value of H cannot be definitely determined before the proposed plant is completed. However, after all preliminary investigations have been made in regard to such a plant, experience has shown that the cost of the plant completed and ready for operation can be predetermined very closely. At least the permissible investment should be known within reasonable limits after all the factors affecting the sale and generation of power have been investigated. If it is impossible to estimate even approximately the actual cost of the plant, then the value of the permissible investment should be used for H.

Let it now be assumed that all the factors entering into the cost of a proposed plant are known with the exception of the cost of the high-pressure pipe. This may be roughly estimated at first, to be determined more definitely after a computation of the diameter has been made, using the above rough estimate. For instance, let it be assumed that the estimated cost of a plant is $90 per horse-power, $15 of which is the estimated cost of the pipe line. However, upon computing D on this basis, and then computing the cost of the pipe for this value of D, it is found that the cost of the pipe will be $7.50 per horse-power, instead of $15 as first estimated. Hence in using $90 per horse-power instead of $82.50, which is very near the true value, an error of about 1.3 per cent. was made in the computation of D. Since the cost of the pipe varies as the square of the diameter, an error of approximately 1.7 per cent. was made in computing its cost on basis of $90 per horse-power, instead of $82.50. But a variation of 1.7 per cent. in the computed cost of the pipe (D being determined on a basis of $15 per horse-power), when taken in connection with the total cost of the installation per horse-power, is scarcely appreciable, and is inappreciable when substituted in equation (13) for computing D. Hence, as in determining the proper value for f, two computations are all that are necessary. First, estimate as near as possible the cost of the pipe. Use this value in connection with the other factors entering into the total cost of the proposed installation for H. Determine the value of D on this basis; knowing D determine the cost of the pipe. If this computed cost is reasonably close to the estimated cost, no change need be made in the value of H. If it is not, then change H to correspond to this new value of the pipe cost and recompute the value of D.

For the purpose of making an application of this formula the following hypothetical case will now be assumed: Let $h = 1800$; $O = 30$; $f = 0.005$ (for first computation); $H = 100$; $c = 0.11$. Two different forms of pipe will be assumed, lap-welded for the high pressure, and riveted pipe for the lower pressures. Let safe tensile stress for lap-welded pipe equal 8,000 lb. per square inch, $= s'$, and, safe tensile stress for riveted pipe equal 10,000 lb. per square inch, $= s''$. The efficiency of the riveted joints will be taken at 70 per cent.

With the above assumed static head and quantity of flow the capacity of the plant will be approximately 4,500 h.-p., and its total cost $450,000 at $100 per horse-power. The total length of the pipe line will be assumed to be 5,000 ft., and its cost, erected, $45,000; one-tenth of the cost of the plant. In making this computation the method of procedure will be as follows: The length of pipe subtended between the static heads of 1,800 and 1,500 ft. will be given a constant diameter and definite length; between the static heads of 1,500 and 1,250 ft., also a constant diameter and definite length, and so forth; decreasing the difference of elevation between the terminals of pipe lengths having fixed diameters as the elevation increases. In computing D by equation (13), the mean static head upon sections of fixed diameters will be taken, and also in computing the cost of these different sections the mean static head will be used. This will result in no error in cost computation, since the thickness of the pipe is a direct function of h where D is constant. The diameter of the pipe will first be computed on a basis of f equals 0.005, and pipe cost equals $45,000. This will be designated D'. Having D', the true value of f may be obtained closely from the tables, and also very near the true pipe cost may be computed. Using these values of f and of pipe cost, D will be recomputed.

Table I. Values of Different Functions for Various Heads.

h	l	D'	V	C	f	D'
1,800 to 1,500	450	1.68	13.5	$9,500	.00453	1.66
1,500 to 1,250	500	1.73	12.7	$9,300	.00450	1.698
1,250 to 1,000	300	1.78	12.0	$4,850	.00449	1.75
1,000 to 800	400	1.84	11.2	$5,550	.00447	1.807
800 to 600	400	1.90	10.6	$4,600	.00445	1.872
600 to 400	600	2.00	9.5	$5,500	.00440	1.96
400 to 250	500	2.08	8.8	$3,730	.00437	2.042
250 to 150	700	2.23	7.7	$3,700	.00431	2.174
150 to 50	950	2.47	6.3	$3,070	.00424	2.41
50 to 0	200	3.00	4.3	$ 240	.00406	2.917

Substituting in equation (13) the values assumed for this hypothetical case: $D' = \sqrt[7]{}\ (63{,}100 \div h)$, for the lap-welded pipe, and $D' = \sqrt[7]{}\ (55{,}400 \div h)$ for the riveted pipe. Also from equation (7) $C = 0.00455\ D^2\ h\ l$, for the lap-welded pipe and $C = 0.0053\ D^2\ h\ l$, for the riveted pipe.

In the accompanying Table I the computations are tabulated in the order in which they were made. The first six sets of values are for lap-welded pipe and the remainder are for riveted pipe. The total cost of the pipe as given in the first computation is $50,040. This value is so near the estimated cost of the pipe that no change need be made in H for the second computation of the diameter. Hence in recomputing D the original expression was changed only by the values of f as given in the above table, instead of the constant value 0.005, as first assumed.

The Stone Crushing Plant at the Menard, Ill., Penitentiary.

The Illinois State Highway Commission has, as one of its functions, the regulation of the distribution of crushed stone produced by convict labor at the State penitentiaries. Under regulations formulated by the Commission, this stone is supplied free to township or county highway officials for use on the public highways, the local officials paying the freight charges. It is also used by the Highway Commission in the construction of experimental roads.

When the act creating the Highway Commission was passed, in May, 1905, there was, at the Southern Illinois Penitentiary, at Menard, on the Mississippi River, about 70 miles below St. Louis, a stone crushing plant consisting of a No. 6 old-style Allis-Chalmers crusher, run with a 90-h. p. Westinghouse engine and an O'Brien boiler. This plant, which is operated entirely by convicts, is within the prison stockade and has a capacity of about 350 cu. yd. per 8-hr. day. As soon as the provisions of the State Highway Commission act became widely known, "free macadam" was in such demand that this plant became entirely inadequate. Accordingly, in the spring of 1906, the Commission, in accordance with its powers as defined in the convict labor act, called on the Board of Prison Industries to install additional crushing plants at the Menard and Joliet penitentiaries. The Joliet plant, which has no unusual features, was put in operation in a few months. At Menard, a side-hill installation was required and it was decided to construct storage and car-loading pockets in connection with the crushing plant. This plant has been completed and put in operation only recently. The combined capacity of the plants at the two prisons amounts to about 1,350 cu. yd. per day.

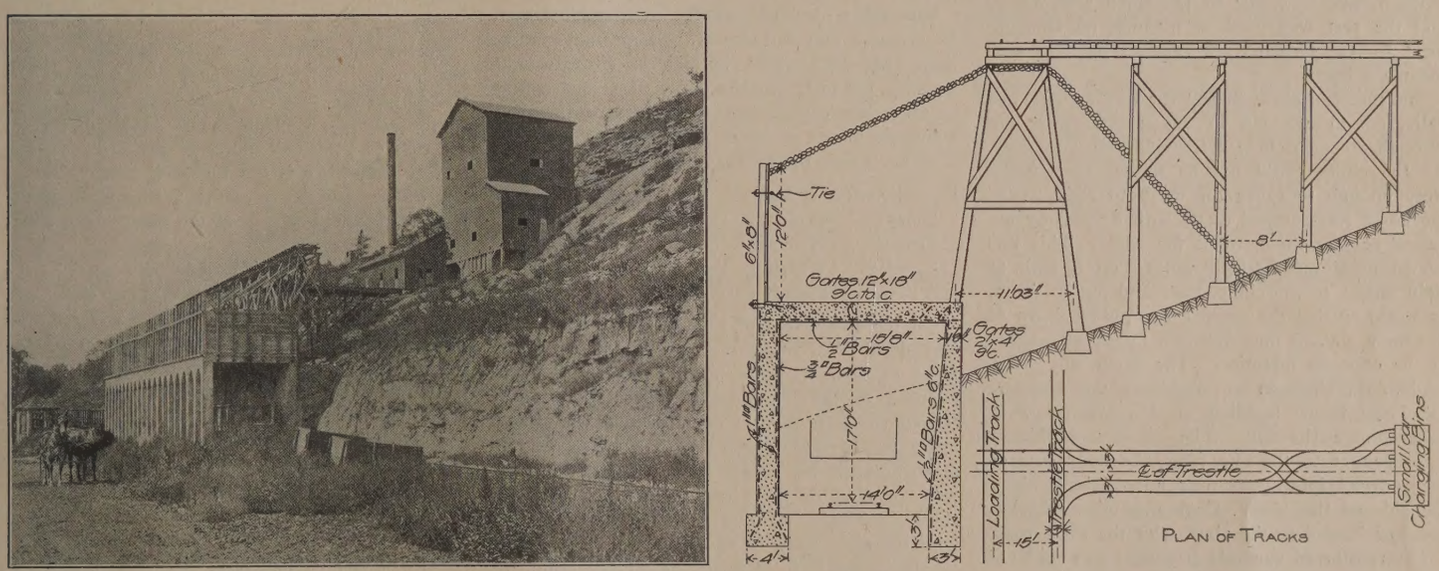
Among the considerations that influenced the design of the Menard plant was the fact that it is desirable to employ convicts as steadily as possible. The preparation of crushed stone for road construction, at a plant without storage facilities, can be carried on economically only during the summer and fall when the crushed stone can be loaded directly from the crusher bins into cars and shipped to the work. Even during the working season it seemed likely that the men would often be idle because of lack of cars or delays on the construction work. With storage facilities, however, the crusher can be run more steadily and better advantage can be taken of empty cars when they are available. It is thought that the present storage pockets at Menard will be practically equivalent to another crusher like that already installed and, as will be seen later, the storage capacity can be greatly increased with very little difficulty.

A new quarry at Menard has been opened up in the rocky bluff, which is characteristic of the east bank of the Mississippi River from St. Louis to Cairo. Just north of the prison yard this bluff rises on an angle of nearly 45 deg., from the river bank to a general height of about 250 ft. above the water. The rock formation, which is a fairly good limestone, is overlaid on top of the bluff with considerable clay soil, a great quantity of which has been gradually washed down over the bluff, reducing the steepness of the incline and necessitating some difficult excavation before solid foundations can be secured. At the foot of the bluff, near the water's edge and parallel to it, a bench about 50 ft. wide is maintained, on which are the tracks of the Missouri Pacific Ry. and the Illinois Southern Ry., the latter being on the river side. Just inside the railroad tracks and about 15 ft. above them, is a county highway, the east fence of which is the west boundary of the State property available for the crusher site and quarry.

The crusher, which is an Allis-Chalmers, rotary No. 7½ machine, is located on the side hill with its head at the elevation of the quarry floor, which was placed at El. 167 on the advice of Dr. H. Foster Bain, State Geologist, whose examinations indicated that the stone below this elevation is of a poor quality. The crusher has a capacity of 500 cu. yd. per 8-hr. day and is driven by a 100-h.p. Chandler & Taylor engine supplied with steam by a 150-h.p. Allis-Chalmers boiler. The engine and boiler rooms are placed on the north side of the crusher building, the engine-room floor being at El. 149.3 and the boiler-room floor at El. 134.8.

The skip cars loaded with uncrushed rock are run downgrade to the crusher and their contents delivered by gravity. The crushed stone passes by gravity from the crusher to the screen and from the latter falls into three small bins, built over two 3-ft. tracks which extend on a slight down-grade on a timber trestle nearly perpendicular to the general north and south direction of the quarry face. About 50 ft. from the crusher house the two tracks on the outgoing trestle diverge and turn off on a single-track trestle whose general direction is parallel to the quarry face. The rails of the single-track or storage trestle are about 27 ft. above the surface of the hill directly beneath them. The trestle legs on the up-hill side rest on concrete pedestals, while those on the down-hill side rest on a reinforced concrete structure, which partially supports the storage pile and under which the railroad cars are run to be loaded. Small dump cars, operated by hand on the trestle tracks, deliver the stone from the three small receiving bins to the storage pile under the high single-track trestle.

The wall on the east or quarry side of the loading track supports one side of the floor of the storage bins; it also acts as a braced retaining wall, supporting the material on the hillside. It is 22 ft. high, 3 ft. wide at the base, and 16 in. wide at the top and is reinforced near its free face with ½-in. vertical twisted bars spaced 6 in. center to center. On the river side of the loading track the weight of the stored material is carried by series of 15x24-in. reinforced concrete columns. These columns are spaced 6 ft. on centers and each rests on a concrete pedestal 4 ft. square and 3 ft. deep. Each column is reinforced with four 1-in. twisted bars, one in each corner, and these are hooped at 18-in. intervals with No. 8 steel wire. The columns carry a reinforced concrete girder 24 in. wide and 37 in. deep, including the 20-in. floor slab of the storage bins. The girder is reinforced with four ¾-in. bars placed in a plane just above the crowns of the soffits of the arches. No arching effect was considered in the preparation of the designs, the arches being introduced solely for their appearance. The 20-in. floor slab is reinforced transversely with 1-in. bars, 4-in. center to center, and longitudinally with ½-in. bars, 8-in. center to center.



Stone Crushing Plant at the Southern Illinois Penitentiary, Menard, Ill.

The outside walls of the storage bins are made of 2-in. planks nailed to 6x8-in. posts, the feet of which are placed in niches along the edge of the floor slab, where they are secured by rag bolts and a continuous waling piece. A second waling strip is secured to the posts near the top of the bins, and provision has been made for securely anchoring the posts against the outward pressure of the retained material by ties running to deadmen buried in the hillside. The crushed stone is dumped from both sides of the longitudinal trestle track, and the pile rests partly on the concrete floor and partly on the hillside. The different sizes are kept in separate piles by light plank partitions. Most of the stone is admitted to the railroad cars through 12x18-in. gates in the floor of the bins over the center line of the loading track, spaced 9 ft. on centers. When the supply of stone is so low that it does not run freely through these gates it can still be delivered to the cars by gravity and without shoveling, through 2x4 ft. gates in the side wall, spaced 9 ft. on centers.

The switch track to the storage pocket leaves the Missouri Pacific Ry. about 1,300 ft. south of

the crusher. It crosses the tracks of the Illinois Southern Ry. and, continuing northward on a rising grade, crosses the county highway at grade and runs along the bluff in a bench cut through the storage plant and several hundred feet beyond. Empty cars are stored on the track north of the crusher and are run under and away from the storage packets by gravity. At present only 125 lin. ft. of concrete structure has been built, 62½ ft. north and 62½ ft. south of the center line of the double-track trestle from the crusher. The length of the storage pockets can be increased readily whenever this seems desirable. It is estimated that the storage pile will contain about 3,500 cu. yd. per 100 lin. ft.

The excavations for the crusher plant foundations and for the loading track amounted to about 1,300 and 8,000 cu. yd., respectively. Though the excavated material was mainly earth, the work was very difficult owing to the great number of buried boulders which from time to time had broken off from the outcrop at the top of the bluff, and rolled down the side hill. Many of these necessitated blasting. All the work was done with convicts, three of whom in general are equal to two ordinary men, and with mules and wheel scrapers. Some of the earth was wasted and the rest was used in forming an embankment for the switch track near its junction with the main line.

The material excavated at the crusher site was delivered to wagons in the switch-track cut through an open chute 2 ft. wide and 2 ft. deep. As the chute could not be placed at a grade steep enough to cause the earth to slide freely, the latter was scraped down with a specially designed scraper made to fit the chute. This was drawn by a mule which walked on a path at right angles to the chute, lines and pulleys being arranged so that the scraper was pulled down as the mule walked one way and up as he walked in the opposite direction. The earth was delivered to the chute in wheelbarrows, and boulders too large to be handled in this manner were rolled down the hill. The crusher machinery was brought to the site from the main line of the railroad in wagons over a road built along the side of the bluff. Coal and other supplies are now hauled to the plant over the same road.

The reinforced concrete structure was built entirely by convict labor and the construction presented no unusual features. The concrete was hand-mixed.

The plans and specifications for the entire installation were prepared by Mr. A. N. Johnson, State Engineer. The plant was built under the direction of the Board of Prison Industries, whose general representative on the work was Gen. J. B. Smith, warden of the Southern Illinois Penitentiary. The storage pockets and the foundations and houses for the crusher, boiler and engine were built by convict labor. Mr. W. S. Gearhart was resident engineer for the Board of Prison Industries during the early part of the work, and Mr. W. T. Beckelhimer during the later part.

A 55-Ton Traveling Tower Crane of unusual working radius has been placed in service on the docks at the harbor of Santa Cruz, on the island of Teneriffe. The tower section, about 20 ft. in height, is carried on six legs, each traveling on a four-wheel truck. The crane member pivoted to the tower is a double-truss structure, 138 ft. long, overhanging 105 ft. from the pivot to the hoisting end and 33 ft. in length from the pivot to the counterbalance. It is designed for a capacity of 55 tons at a radius of 58 ft. from the pivot. The crane is operated electrically, a 54-h.-p. motor being used, both for hoisting and for moving the tower, a 15-h.-p. motor for revolving the crane and a 30-h.-p. for the trolley.

Experiments on Wind Pressure.

About five years ago Dr. T. E. Stanton began a series of investigations to determine the distribution and intensity of the pressure of the wind on structures. This research work was proposed by the Committee of the National Physical Laboratory of Great Britain as the first investigation to be undertaken in the Engineering Department of the Laboratory. The first part of this research was communicated to the Institution of Civil Engineers in December, 1903, and related to the resultant pressure and distribution of pressure on flat plates normal to and inclined to the direction of a uniform current of air.

As those experiments were made in a channel 24 in. in diameter, the dimensions of the plates were necessarily small, but between the range in dimensions obtained, which for circular plates was from ¼ in. to 2½ in. in diameter, the results indicated that the resistance of geometrically similar plates was proportional to the area of the plates. In the case of dissimilar plates, however, such as square plates and long rectangles, the resistances per unit area differed considerably.

The value of the resistance so found in these experiments on small plates was somewhat smaller than that determined by Dines, Frowde and Langley for plates of the order of 1 square foot in area. For comparison, the values of the constant, K, in the pressure velocity relation, $P = KV^2$ are given in the following table:

Experimenter	Method	Value of K
Dines	Whirling Table	0.0029
Frowde	Moving Carriage	0.0037
Langley	Whirling Table	0.0033
Author	Plate in uniform current	0.0027

On the completion of this part of the work, it was decided to make observations on flat surfaces of areas ranging up to 100 sq. ft. when exposed to the wind, since general experience tended to show that in actual winds whose velocity was not uniform over time or space, the mean pressure per square foot on a large surface was considerably less than that on a small one. As a knowledge of this variation in resistance with the dimensions of the structure, if it exists, is all-important in design, the investigation of this problem was made the chief feature of the new experiments. This work was described by Dr. Stanton in a paper read before the Institution of Civil Engineers on Dec. 3, of which an abstract follows.

For the purpose of the work a steel windmill tower was erected in the grounds of the National Physical Laboratory at Teddington. The experimental boards and models of structures were attached to a light framework carried by the cap of the tower, the height of the center of the boards from the ground being 50 ft.

After some preliminary experiments, the method of observation finally adopted was the determination of the constant in the pressure-velocity relation for pressure-boards of varying dimensions and for the models of structures. It was found, as anticipated from a knowledge of the variable character of the velocity of the wind, that single observations were quite worthless for the purpose in view, but that, if for any pressure-board or model about 200 observations of the velocity of the wind and the corresponding pressure on the board were taken, it was possible to obtain a fairly accurate value of the constant. In these observations the velocity of the wind was estimated from a pair of pressure-tubes, similar to those used by Mr. Dines in his anemometer, placed about 15 ft. above the center of the board. These tubes were connected by lead pipes to a sensitive water-gauge, of the type used in the author's previous experiments, placed at the foot of the tower. The resultant pressure of the wind on the board was estimated from a measurement of the pressure produced in a closed cylinder of air by the deformation of a thin steel diaphragm forming its cover which was in contact with the center of the pressure-board. This pressure was also transmitted through lead pipes to the foot of the tower, and there measured by a similar tilting gauge to the one used for the velocity estimations. The simultaneous observations of pressure and velocity were only possible in the short periods of time in which the velocity of the wind was fairly constant. Such periods, lasting from 2 to 5 seconds, were found to occur about once a minute in a fairly steady breeze.

The results of these observations on three pressure-boards, one 5 x 5 ft., one 5 x 10 ft., and one 10 x 10 ft., gave practically identical values of the constant in the pressure-velocity relation. In units of pounds per square foot and miles per hour, the mean value of this constant for the three boards was 0.0032. As this value agreed so well with the average of those obtained by previous experimenters when using plates of the order of 1 sq ft. in area, it was not considered necessary to make experiments on plates smaller than the one 5 x 5 ft. in the present case.

Further observations on the intensity of the pressure at the front and back of the boards appeared to show that the cause of the higher value of the constant compared with that obtained in the case of the small plates in the 24-in. experimental channel, was the relatively greater intensity of the negative pressure at the back of the boards compared to that at the back of the small plates.

Experiments were also made on a model of a braced-girder 29 ft. long by 3 ft. 7 in. deep, and on a roof model whose sides were 8 x 7 ft. The ratio of the resistance per unit of area of the model girder to that of a square board in the wind was found to be precisely the same as the ratio of the resistance per unit of area of a small model of the girder made to a linear scale of 1 in 42 to a square plate in the experimental channel and uniform current used in the previous experiments.

The resultant pressures on the roof were obtained, for both windward and leeward sides, at angles of 30, 45 and 60 deg. inclination to the horizontal, and indicated the considerable suction effects on the leeward side of a roof when the pressure inside the building is augmented from the windward side by open doors or windows.

These results lead to the conclusion that the resistance of a complicated structure in the wind can be accurately predicted from a determination of the resistance of a small model of the structure in an experimental channel.

The Motor Boat for naval service appears, from discussions at the recent meeting of the Society of Naval Architects and Marine Engineers, to have some advantages over the steam launch. The question of reliability of the gasoline engine as a prime mover, which has heretofore deterred its more general adoption for naval service, has been practically settled by elaboration of design and refinements in construction, so that the general opinion is that when a well-built engine is placed in the hands of a man trained for the work, it is sufficiently reliable for naval use. It was pointed out in the discussion that the launch engine is not subjected to the jar and vibration that attend the operation of the automobile motor, and that the latter engines have now been made so flexible and reliable that 100-mile sealed-bonnet tests are possible. The saving of space required for the machinery over that of the steam launch is 20 to 25 per cent., according to the design of the engine.

The Gatun Dam.

The annual report of the Isthmian Canal Commission for the past year, and the issue of the "Canal Record" for Dec. 4, together give some instructive information concerning the Gatun dam, which is the crucial feature of the Panama Canal design. In one of the appendices to the Commission's report, Mr. Ernest Howe, geologist, furnishes a detailed account of the geology of the site of this great structure. According to his statements, the sides of the broad lower valley of the Chagres at Gatun converge and the stream passes through a gap in the mountains which is a little over one mile wide. The geological conditions at the site of the dam are the result of many changes. At some time after the Gatun beds had been deposited, the central and southern portions of the present isthmus were subjected to volcanic disturbances and the whole region underwent considerable deformation. The geology of the central portion of the isthmus indicates that a vast amount of erosion took place following the elevation of the land, and that the sedimentary and later volcanic rock now exposed represent but a very small part of the bed that originally covered the region. There were three great uplifts which threw these beds out of position and the third resulted in the streams intrenching themselves in deep canyons in the material left by the second uplift. When the Chagres River had succeeded in cutting somesomething more than 200 ft. below the second level, the land began to sink and the gorges were gradually filled with gravel, sand, silt and clay. It is believed that this sinking continued until a large part of the Colon level now represented by swamps, was submerged and covered by sea water. Finally there was a slight elevation of the whole region, which raised these deposits a few feet above tide level.

During the periods of maximum uplifts, when streams were flowing in canyons like those of the Chagres, the rivers were heavily charged with detritus. All of the finer part of this material was swept down to the sea, the coarsest was deposited along the upper portions of the stream, but gravels and small boulders filled the stream beds to the mouths of the rivers. Where subsidence took place before the period of canyon cutting has ceased, as in the case of the Chagres, the velocity of the streams was checked and they were capable of carrying only fine material, which was deposited far back from the mouths. As a result, in the bottom of such a gorge as that of the Chagres at Gatun, there is a comparatively thin deposit of coarse gravel, sand and a few boulders, while the greater part of the alluvium is of the finest clay and silt, with a certain amount of fine gravel and sand mixed with it.

The boring records indicate considerable sand and gravel at points comparatively near the present surface; Mr. Howe considers that it is unlikely, however, that any sand or gravel, unmixed with clay, occurs in the deposits except at or very near the bottom. The reason that the boring records specify "Sand and Gravel" is that when the samples are taken from time to time during the process of sinking the hole, only the coarsest material is collected, the finer clay being held in suspension and carried off with the water flowing from the hole. In certain of the holes temporary flows of water were encountered, but after a few hours these invariably ceased. They are believed to occur when, in the course of making a boring, the casing is introduced into a lenticular deposit of sand or water-bearing gravel under pressure from the overlying bed. The sands, being surrounded by much less pervious material, are in the nature of reservoirs in which water is stored under pressure, and when this pressure is released at the point where the casing enters the sand, water may rise to the surface if under sufficient pressure and flow from the top of the casing until the pressures are readjusted. Professor Howe believes that far from indicating porous materials underlying the site for the dam, the occurrence of such flows of water only proves the extremely impervious character of the materials lying between the surface and the water-bearing beds. As a foundation for an earth dam, he considers that the geological facts show that the alluvium filling the valley of the Chagres at Gatun will be entirely satisfactory.

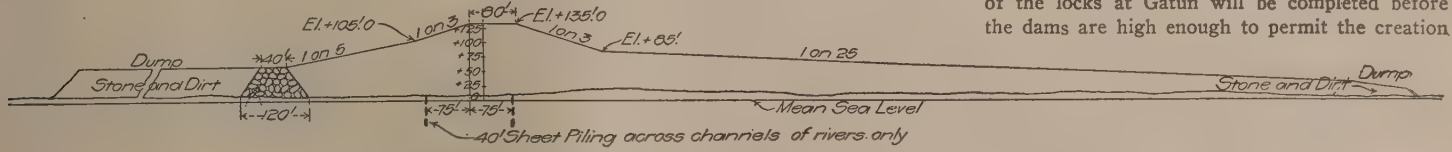
The great locks will be at the northeast end of this dam. They will be almost entirely in fine-grained argillaceous sandstone, except at the lowest point to be reached at the extreme southern end, where conglomerate occurs beneath the sandstone. The sand of which these rocks are composed was derived from the older igneous rocks of the region. The debris resulting from the breaking down of these rocks was transported by the streams to the sea and deposited, layer upon layer, until some 200 ft. had accumulated. Considerable clay was mixed with the sand, and this, together with the lime derived from the sea water, acted as a cement and bound the grains of sand together, forming a massive rock.

The rocks from which the sands were derived were almost, if not entirely, destitute of quartz, and the sandstones lacking this mineral are more subject to decomposition than rocks consisting principally of quartz sand. As a whole the rocks are stated to make excellent cores with the diamond drill. When taken out and exposed on the surface they remain firm and hard; in rare instances they have been found to crumble and break down into a sandy clay. The rocks are well compacted and capable of supporting heavy loads when confined, but being poorly cemented are unable to withstand erosion, and should be fully protected where such action is anticipated On account of the very considerable amount of clay present in all of the rocks, it is believed by Mr. Howe that they will prove to be almost entirely impervious to water, except in the case of the conglomerate. Wherever the latter is encountered, it may be expected to be water-bearing at all times. This rock stands in vertical walls without timbering in a test pit sunk into it, and so long as it is confined it will support heavy loads.

The section of the Gatun dam has recently been slightly changed, and the accompanying diagram shows the revised form. The method of building the dam and filling the lake is explained in the "Canal Record" substantially as follows:

The total length of the dam is about 7,700 ft. from the locks on the northeast to the hills on the southwest. At about its middle point there is rising ground, now known as Spillway Hill, where the regulating works are to be built. It is proposed first to dig a channel about 300 ft. wide through Spillway Hill, the bottom of it to be at sea level or thereabouts. The Chagres River will be passed through this channel during the construction of the dam across the valleys on either side of the hill. As soon as the portions of the dam abutting on Spillway Hill are high enough to hold from 50 to 55 ft. of water in the lake it is proposed to build across the channel through Spillway Hill a concrete dam high enough to hold the lake at this level. This will be done to facilitate the work of dredging in the Chagres Division. During the dry season following the completion of this dredging the dam across the channel through Spillway Hill will be brought to its full height, and a permanent spillway constructed, which will include the necessary regulating works, by means of which the surplus water of the lake will be passed down to the level of the sea.

Preparations will be made for closing the channel through Spillway Hill before the Chagres is turned into it, and all closures will be accomplished during the dry season, thus enabling the Chagres to form a lake to the height of the dam across the spillway channel during the following wet season. It is considered probable that Gatun Lake will not be allowed to fill to the height of 50 or 55 ft. until the upper end of the lock at Gatun has been completed and the upper gates have been erected. If this plan of procedure is carried out and the progress of the work comes up to anticipation, the upper portion of the locks at Gatun will be completed before the dams are high enough to permit the creation of the lake at 50 to 55 ft. elevation. This lake will make water transportation available for handling large quantities of timber that will be submerged by the lake, and contracts may be let for cutting this timber, or it may be sold standing to the highest bidder. The exact capacity of Gatun Lake has not yet been known. The total amount of excavation at the Gatun lock, dam and spillway up to Dec. 1 has been 1,136,000 cu. yd.



The Revised Cross-Section of the Gatun Dam of the Panama Canal.

A BRICK CHIMNEY under construction for the 59th St. station of the New York Steam Co. lost a small section of its top in a hurricane on Dec. 14. The chimney is being built of perforated radial brick by the M. W. Kellogg Co. and will be 264 ft. high and 17 ft. in diameter at the top. The condition of the chimney at the time of the accident is stated by Mr. M. W. Kellogg, president of the M. W. Kellogg Company, to have been as follows: "On Friday, the day before the storm, the brickwork of the chimney had been completed to within about 12 in. of the cap at the top, the chimney wall having been carried up about 2½ ft. on that day. Practically no work was done on Saturday morning, owing to the snow, which later turned to rain. Upon increase of the wind in the afternoon to a gale of 70 miles velocity, the brickwork freshly laid at the top on the afternoon before was dislodged and fell. A careful investigation indicates that none of the brickwork but that laid on Friday and which was still green was affected. The height of wall dislodged was from 2 to 2½ ft., but, owing to the circumference of the chimney at the top of about 55 ft., the quantity of material that fell was large, probably 20 tons." When the top of the stack fell it crashed through the roof of an adjoining building, killing one person.

Accuracy of Slide Rule Computations.

By John Berg.

When preliminary estimates on engineering works are of such a nature that the slide rule can be used in their computation, then this little instrument ought to be employed. It not only saves considerable time, but it saves much mental effort; and the final results are more apt to be free from large errors than when all the work is done the long, tedious way. To sit day after day for seven or eight hours, with only a short intermission at noon, multiplying and dividing, and not doing much else, is very tiresome work. I have also found that I am more liable to errors during the latter part of the afternoon after such a day's work.

I have known the average end-area formula to be used in finding the volume of a prismoid; the unit weight per cubic foot assumed to the nearest tenth of a pound; the stress in full members computed to the nearest pound and better. More consistent would it have been to have used the prismoidal formula in finding the volume; assume unit weight per cubic foot to the nearest pound; compute stresses to the nearest hundred pounds, and do all the work of multiplication and division on a slide rule—i. e., by graphic logarithms. There is much less chance for a large error in the latter result, and it should be obtained with less fatigue and greater speed.

Practical opportunity has recently been afforded me to test the accuracy of slide rule computation, especially in the reduction of cubic feet to cubic yards, and, though having already had a good deal of respect for the efficiency of the slide rule for accuracy on computations, I was more than satisfied with the results obtained in the investigation. Omitting one large error, apparently due to a slipping of the slide and reading before re-setting, and instantly discovered on a rapid and rougher check of the work, the following results in division were obtained:

Of 28 three-figure quotients, ranging in magnitude from 391 to 999, inclusive, 23 were correct to the nearest unit; the remaining 5 were each 1 unit too small. However, the maximum "unit" error in the case of the 5 quotients was really only ⅔ and not 1; or, reduced to a percentage basis, the maximum error was 0.11 of 1 per cent. The sum of the 28 numbers representing cubic feet was 510,349; the sum of the 28 quotients as obtained by the slide rule was 18,900 cu. yd., a sum that is too small by 1.8 cu. yds.; i. e., the sum as obtained from slide rule results is in error by less than 0.01 of 1 per cent.

A similar study was made of 116 quotients ranging in magnitude from 1,007 to 8,590, inclusive, the numbers being pretty well distributed under the respective groups given in the accompanying table.

With the exception of group 9, maximum and total errors had to be added to the corresponding slide rule result to give the correct amount.

Of the 144 quotients only 2 were in error by as much as 0.1 of 1 per cent,. and these were 0.110 per cent. and 0.101 per cent. respectively. From the table it will be seen that in no case is the average error in the sum of even a comparatively few numbers one-half of this maximum. The average error for the 144 quotients was 0.024 of 1 per cent., or, expressed as a ratio, 1 in 4,200, a result quite comparable with that obtained with a steel tape over moderately rough ground. By a comparison with the results of the thorough investigation of the relative accuracy of planimeters by Prof. Lorber, of Loeben, Austria, quoted in J. B. Johnson's "Surveying," it will be seen that the slide rule is far more accurate than the polar planimeter, and that it compares very favorably with the suspended and rolling planimeters, even though the latter of these is considered an "instrument of precision."

Assuming the possibility of absolutely accurate horizontal measurements, it will not be very difficult to find a field engineer who will admit the possibility and even the probability of errors of 0.1 of a foot in the differences of level elevations in a cut or fill. If the cut is of an average depth of 10 ft. the above error is 1 per cent. of the quantity. Cut the probable error down to 0.01 of a foot and the chances are 70 to 1 that the slide rule is more accurate still. Allowing, further, for errors in plotting up the field notes; errors in the transfer to a tracing; errors due to shrinkage of tracing cloth and blue-print paper; "personal equation" errors in following contour lines with the tracing arm of the planimeter; errors inherent in the planimeter measurement itself, allowing for all these and other probable errors, not more than due credit will be given if we say that the slide rule is at least as accurate for the conversion of field data into office estimates as are the methods used in obtaining these data by the average assistant in the field and in the drafting-room.

Table of Errors in 116 Quotients.

Group.	Number and Range of Quotients. Correct.	Small.	Large.	Total.	From.	To.	Numerical Error. Max.	Total.	Per Cent. Error. Max.	Average.
1	23	5	0	28	391	999	0.7	1.8	0.110	0.010
2	25	14	5	44	1007	1980	1.4	12.5	0.095	0.020
3	11	8	5	24	2000	2944	1.9	3.7	0.068	0.007
4	4	5	2	11	3027	3934	3.9	9.1	0.101	0.024
5	0	9	1	10	4033	4991	4.4	20.0	0.088	0.044
6	2	6	0	8	5105	5945	2.5	9.5	0.042	0.021
7	2	6	0	8	6070	6991	4.8	14.6	0.075	0.028
8	0	6	1	7	7131	7989	6.7	25.5	0.091	0.047
9	0	2	2	4	8137	8590	3.3	0.6	0.040	0.002
Total..	67	61	16	144	391	8590	6.7	96.1	0.110	0.024

Should it be desirable to approximate to greater accuracy than is possible with a direct reading of the rule, then the first figure may be obtained mentally when a series of numbers is to be divided by the same small number as, in this case, 27. In such a series of 21 quotients, also taken from practical work, only one was in four figures, all the others having five figures and ranging in magnitude from 11,324 to 69,493. Of the 21 quotients 5 were correct to the nearest unit; 1 was too large by 1 unit; 1 was too large by 2 units; 7 were too small by 1 unit; 3 were too small by 3 units; and 4 were too small by 4, 5, 6 and 7 units each. Reduced to a percentage basis the maximum error in this series was 0.022 of 1 per cent., and the average error for the 21 quotients was less than 0.004 of 1 per cent., 1 in 25,000! If these errors are larger than first expected it can be explained by the fact that the maximum was obtained from a quotient beginning with 2, which was followed by a 7. The four largest errors of 4, 5, 6 and 7 were all found in quotients whose first digits were smaller than the following digit. But for this fact we might have expected even a better ratio for the average error than 1 in 25,000. Though the observations are not numerous enough to warrant enunciation of a proposition to the effect that the percentage of error is independent of the size of the first digit, yet this seems to obtain, as might be expected, though it appears somewhat erratic.

As in the case of straight division on the rule, its use in getting the remaining figures, after the first has been obtained mentally, has yet the advantage of speed and accuracy with less liability to large error. The probable percentage of error in this method depends mainly on the relative numerical values of the first two figures: e. g., if the first figure is 1, obtained mentally, and the second 9, obtained on the scale, then the maximum error on direct reading of, say, 0.1 per cent. may be reduced to 0.05 per cent. If, on the other hand, the first figure is 9, and the second figure is 1, the above maximum of 0.1 per cent. on direct reading will approach 0.001 per cent., a total range in the ratio of probable maximum error from 1 in 2,000 to 1 in 100,000, accepting an ordinary maximum error of 0.1 per cent. for direct reading on the scale.

Since accuracy in reading depends primarily on the accuracy of the graduations and secondarily on close interpolation, we should expect as small errors in multiplication as in division. This is well illustrated by the following example: Of 27 two-factor, four-figure products, ranging in magnitude from 3,321 to 8,046, inclusive, none ending in 0, 7 were correct to the nearest unit; 2 were too large by 1 unit; 12 were too small by 1 unit; 2 were too small by 2 units; 1 was too small by 3 units; and 3 were too small by 4 units. Reduced to a percentage basis, the maximum error was 0.079 per cent., and the average error was 0.019 per cent., both of which results compare favorably with the maximum and average errors for quotients.

For most engineering computations with the pocket-and-office rule it is a useless waste of time to attempt reading all results to four significant figures; but if the greatest possible accuracy is desired, result to four figures, beginning with the figure 1, may be read to the nearest unit; and those beginning with 2 and 3 may be read to the nearest 5 units; but in the remaining numbers the fourth figure may as well be 0. Even with these limitations average errors ought to be well within the ratio of 1 in 2,000, while the chances appear very favorable for much better results, unless several settings have been made as in a continued multiplication or division.

Engineering students can never begin too early getting acquainted with a slide rule. Its study can with much profit be begun in the freshman year, or as soon as the study of logarithms is well under way. The two studies are more than supplementary; they are identical. A proper understanding of the one is a thorough knowledge of the other. A slide rule is a whole set of logarithm tables in itself, in which, as in the tables, the characteristic is absent; a fractional part of the scale unit of which is an equal fractional part, the mantissa, of the unit exponent whose base is 10; and on which the engraved figures and intermediate graduations are the numbers corresponding to the fractional exponent, or mantissa, above mentioned.

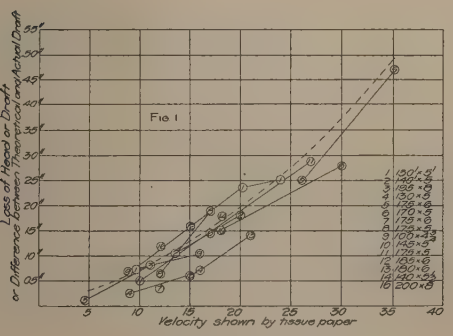
For evident reasons it is not always best to require students to purchase slide rules early in their college course, yet students have been known to be sorry afterwards that they had not been compelled to get the rule much earlier. Unless one has had a rule in use a little so he can himself test it for accuracy, he ought to intrust its purchase to someone better acquainted with its use. For lack of something else which may be better, I have been substituting this test for accuracy in the graduations on the scale most used: Find the reciprocals of several numbers along the whole length of the scale, say all the prime numbers from 11 to 97, inclusive, and as many more similar three-figure numbers, reading their reciprocals to four significant figures. Compare with a table of reciprocals found in most any handbook. With the usual pocket rule the reciprocals will not all be correct to the nearest unit; but the maximum error, with careful reading, ought not to be over 0.1 per cent., and the average at least not more than one-half, and preferably not much over one-fourth, of this maximum, the average being obtained by adding all errors numerically and not algebraically. If these limits are considerably exceeded another rule should be tried, for there is a great difference in the accuracy of slide rules on the market.

The Design of Power Plant Chimneys.

By Frank Kingsley.

The design of a power plant chimney is a problem which is usually solved by rule-of-thumb methods. The common practice is to look over the vague generalities expressed in engineering hand-books and then lay out a structure vastly in excess of the requirements. We say that a chimney 150 ft. in height and 6 ft. in diameter will serve 1,000 h.-p. of boilers, or carry off gas from the combustion of 5,000 lb. of coal per hour, but what is the result if the diameter is made only 5 ft.? What if the height is made 100 ft.? What if 500 h.-p. is added to the plant? What if forced draft is introduced? Answers to these questions cannot be obtained from any of the formulas in common use.

Considering Kent's formula, $H = (0.3 \text{ h.-p.} \div E)^2$, which is the best known of any of the empirical formulæ, it is manifest that if it gave a satisfactory stack for a battery of Babcock & Wilcox boilers (where the gases make three passes through 10 or 12 rows of staggered tubes) it would give, for a battery of Manning upright boilers, a structure vastly in excess of the requirements. This is because the resistance to the passage of gases through the former type of boiler is much greater than through the latter, and more draft is needed to pass the gases through it. It is also evident that with a stack designed from this formula the proper grate area and rate of combustion will have to be determined by experiment after the plant is in operation. In fact, Mr. Kent, in his hand-book, admits that this is necessary.



Draft Measurements.

The Stirling Company, in its book on steam boilers, offered a method of stack design by which the resistance of the boiler, of the bed of coal and of the flue are approximated from past experience, added together, and from this total the necessary height of stack is obtained on the assumption that the effective draft is 80 per cent. of the theoretical draft. The area is found from the formula, $A = 4.92 \text{ h.-p}^{2/5}$.

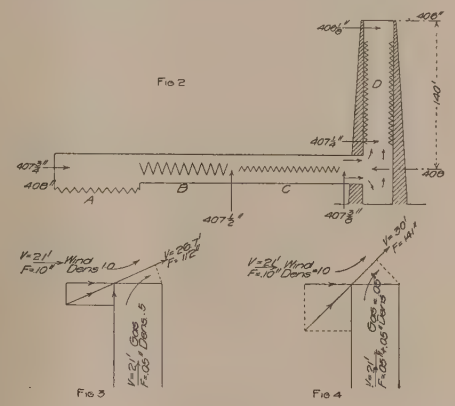
Both of the foregoing methods of stack design are incomplete, because they do not consider the fact that a stack has two duties to perform; first, to produce draft at its base; second, to carry off gas.

Now, if any stack is considered from the standpoint of the quantity of gas carried off, the problem becomes solely one of velocities, because the relative density of stack gas does not vary, at any certain temperature, over 1 per cent, and the variations from the absolute temperature of 1,000° F. found in fairly good practice are negligible. Hence, the weight of gas passed up any stack depends simply upon the speed at which it travels, and it would appear that, if high velocities could be obtained, stacks of extremely small diameter could be used.

However, the resistances within the stack become greater as the velocity of gas increases, and, while diameter may be decreased by high velocity, the effective draft at the stack base will be decreased so that height will have to be added to make up for it, or else grate area increased to make up for the decreased draft.

A question might be asked here as to how the high velocities may be obtained, and it will be necessary to bear in mind that the velocity of gas in any stack depends simply upon the amount of gas delivered to the stack. As more coal is burned, and more gas is produced, the velocity rises; as less coal is burned, the velocity falls. The stack height is absolutely immaterial, except in that it establishes as a limit the point where the entire theoretical draft is devoted to producing velocity, and this condition is, of course, never reached. A stack 200 ft. high with dampers closed tight will give no velocity to the gas in it, notwithstanding the height; while the gas in a 50-ft. stack over an upright boiler with under grate forced draft may nearly reach the maximum velocity of 29 ft. per second.

In order to throw some light on this subject, the writer made a series of observations, at the instance of the Parson Manufacturing Co., upon a number of average-size power plant chimneys. These consisted of stack draft and temperature readings on each stack; and, after each reading, obtaining the velocity of the gas in the stack by timing a handful of tissue paper, introduced into the stack, from the base to the top.



Draft Diagrams.

The temperatures of stack gas and external air were obtained with 800° thermometers, and the draft was obtained with a spiral draft gauge reading to 0.01 in. The paper was introduced into the stack by springing the breeching slightly away from the brickwork where it entered the stack. Its velocity was timed with a stop-watch. The paper was found to fall in stationary air at the rate of, roughly, 2 ft. per second for the average stack; and, consequently, it was assumed, on account of the less dense medium, to show a velocity of 3 ft. per second less than the real velocity of the stream of gas.

Barometer and weather conditions were noted daily, but as the average barometric variation only affected the draft 0.01 in., it was neglected eventually. As all the plants visited had either a damper regulator or ash pit blower regulator, a number of readings were taken with this on and with it off. Each set of readings was averaged; and the variation from the mean noted. This was for the purpose of obtaining an idea of the efficacy of the apparatus, and to detect the presence of unconsidered factors. At the lower velocities of stack gas, or where the stack was under-loaded, the variations were large; but at high velocities they were inconsiderable. Unfortunately the importance of the pressure created by a wind blowing into an open fire-room door was not considered when the readings were taken, and this introduced an error in several instances.

The aspirating effect of wind in blowing across the top of the stack was found to be negligible; in fact, when it is considered that the ordinary "brisk wind" has no greater velocity than the gas in the average stack—namely, 20 ft. per second—it becomes evident that this very common assumption is not based on sound reasoning. It is also evident that no appreciable change of temperature can take place in a properly loaded stack, because the gas travels to the top in 5 to 10 seconds.

It was found that the pyrometer was very slow in action, in comparison with the draft gauge; and, as the conditions were subject to continual slight changes, a certain amount of error was introduced from this source also.

With the above readings, and a knowledge of the stack height, it became possible to calculate the draft which the stack should give at its base; and by subtracting from this the actual reading shown by the draft-gauge, the loss of head for certain velocities in the stack were obtained. The results obtained are given in Fig. 1, being platted with vertical measurements, indicating loss of head, or draft, in inches of water, and horizontal measurements indicating velocities (not corrected for the action of gravity on the tissue paper).

It will be observed that the platted results take the form of a roughly defined curve, regardless of the size of the stack, showing that resistance is not due to friction alone, which is determined by the well-established formula:

$$\text{Loss of head} = CV^2H/D.$$

In view of this fact it might be well to analyze the forces and resistances acting on stack gases, with the help of Fig. 2. Assume a stack about 140 ft. in height, with an atmospheric pressure at its base of 408 in. of water, or 14.75 lb. per square inch. Call A the grate, B the tubes and C the breeching. Let D represent the rough sides, giving a skin friction. Now, with air at about 50° F. and stack gases at 560° F., we may expect that the air between the top and the bottom of the stack will exert a pressure of 2 in. of water, and the gas in the stack (being of half the density) will exert 1 in. pressure. Then at the top of the stack there will be 406 in. pressure, and at the base inside 1 in. more, or 407 in., if the gas is stationary. However, assume the gas is moving, and assume that, just before exit, the gas has ⅛ in. of pressure above the atmospheric pressure (this will be considered later). Then, if the skin friction, D, is ⅛ in., the pressure just above the breeching connection will be 406¼ in. + 1 in. = 407¼ in. Now, to turn the gases from their horizontal direction as they leave the breeching requires a certain force. Calling this ⅛ in. the pressure at the point where the observations of draft were made would be 407⅜ in., and a draft gauge would show ⅝ in. of draft at this point, whereas if the gas were stationary the pressure would be 407 in. and the gauge would read 1 in. From this it will be seen that each loss of draft or head shown on Fig. 1 was composed of three separate resistances: (1) exit velocity, (2) skin friction, (3) change of direction.

To consider each of these separately: When air at a small pressure flows through a frictionless tube into the atmosphere it obtains velocity as it enters the tube and loses pressure. In other words, its potential energy of pressure changes to the kinetic energy of motion. At the exit the kinetic energy or "velocity head" is dissipated in eddies and friction as each particle gives up its velocity to other particles, and if a draft gauge were placed at the exit no reading would be shown. In the case of a chimney, however, the condition is one where a stream of gas of, roughly, half the density of air, is made to issue from the top of the stack, and in so doing it displaces its own volume of the

heavier medium. Assume a frictionless, weightless piston, introduced momentarily just below the top of the stack; on its under side it will have the force due to velocity of the gas, and equal to $h = v^2/64.4$, and on its upper side it has air, to move which at the same velocity requires a force $H = v^2/64.4$; but since the density of air is twice that of gas, the force or column, H (although the same height), weighs twice as much as h; consequently, twice the force, which the velocity of gas exerts alone, is needed to move the piston at the velocity of the stack gas.

Another method of explaining this would be to consider a stack 1 ft. square, as shown on Fig. 3. Assume the velocity of the gas in the stack, as shown, to be 21 ft. per second, which gives it velocity head of 0.05 in. Then assume a wind blowing across the top of the stack at the same velocity, which would give it (due to greater density) a velocity head of 0.10 in. By completing the parallelogram of forces, it is seen that the two forces will combine and proceed at an angle of 26° 30′ from the horizontal; and, as the mean density, upon which the resultant force of 0.112 in. acts, is .75, the velocity in that direction will be 26.7 ft. per second. But since the sine of 26° 30′ is 0.446, it will be necessary to have the same amount of gas that passes through an opening 1 ft. square at 21 ft. per second velocity pass across an area of 0.446 sq. ft. at a velocity of 26.7 ft. per second. This is manifestly impossible, and the pressure in the stack must be increased or the stack velocity decreased.

If the same conditions occur with 0.05 in. excess pressure in the stack, Fig. 4, the two forces combine to produce 0.141 in., which acts at 45 deg. from the horizontal and gives a velocity to the combined gases of 30 ft. per second, as they have a mean density of 0.75. Now, the sine of 45 deg. is 0.70; so that we have to pass gas at 30 ft. per second across an area of 0.70 sq. ft., which makes a discharge of 21 cu. ft. This is the exact amount of gas that the stack is carrying.

Hence a pressure must be exerted at the top of the stack equal to the difference between that which will move the gas and that which will move the air at the same velocity, or, roughly, $R_1 = \frac{1}{2} CV^2 \div 64$, C being the factor for changing feet of air at 60° to inches of water; or, corrected for gas at 500°, this amounts to $R = 0.00011V^2$. The second of the resistances, or the skin friction of the stack, is unquestionably expressed by the formula $R_2 = CV^2H \div D$, C being a constant depending on the nature of the substances in contact and the density of the gas. Sturtevant's factor, which is very commonly used for the friction of the air in iron pipe, would give, allowing for the decreased density, $C = 0.00000285$; and since it is generally conceded that this is 75 per cent. of the factor for brick ducts, the resistance to skin friction would become $R_2 = 0.0000038V^2H \div D$.

Now, the value of $H \div D$ varies from 15 to 35 for average stacks, with a mean of about 25; and by substituting:

Maximum ..... $R_2 = 0.000133V^2$
Mean ..... $R_2 = .000095V^2$
Minimum ..... $R_2 = .000057V^2$

The third resistance, or that caused by the bend at the base of the stack, is one to which it is difficult to ascribe an exact value. It would seem from Fig. 2 that the velocity head should be neutralized by eddies due to impingement of the gas upon the perpendicular stack walls. The resistance of the bend would then be the head necessary to again impart velocity to the gas, or $R_3 = 0.000115v^2$.

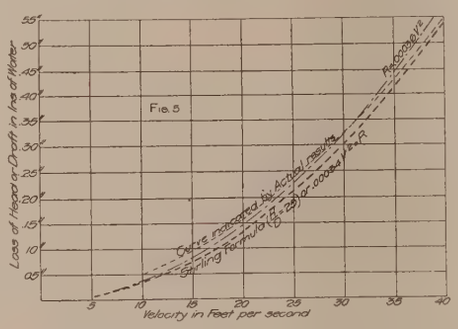
In several cases this resistance has been found to agree with the formula $R_3 = 0.00015v^2$; and this, at 18 ft. velocity, amounts to 0.05 in., or the amount allowed by Stirling for any bend, the velocities given by their formula being about 18 ft. per second. If for want of a better established formula this be taken, the total resistances are as follows:

Maximum ..... $R = .000393V^2$
Mean ..... $R = .000355V^2$
Minimum ..... $R = .000317V^2$

Now, even with the high velocity of 30 ft. per second, the variation from the mean is only 0.034 in., and this, when compared to the draft of the stack, or, say, 1 in., is negligible. Therefore, $0.000355v^2$, or, say, $0.00036v^2$ can be taken as a close approximation of the loss of head in any stack, where V equals velocity of stack gas in feet per second.

Stirling uses $\Delta D \ fW^2VH \div A^3$ as a formula showing the loss of head or draft in stacks "due to inertia and friction." While this formula is incorrect in form, the constant used has been undoubtedly determined by the average of a large number of experiments, and for average conditions it may be used as a check. By substituting for A its equivalent, $\frac{1}{4}\pi D^2$, and for C its equivalent, πD, and by using the mean value for f, or 0.0017, the formula becomes:

$$\Delta D = 0.0018H \cdot (W/A)^2(4\pi D/\pi D^2).$$



Velocity and Loss of Head Curves.

But the weight of any sample of flue gas, or W, at average temperature varies but little from (volume $\div$ 23), and volume divided by area equals velocity. Substituting and cancelling:

$$\Delta D = 0.0000136v^2H \div D;$$

Then calling $H \div D = 25$, as before:

$$AD = 0.00034v^2.$$

This checks closely with the previous formula:

$$R = 0.00036v^2.$$

The curves in Fig. 5 are platted from these two formulæ, and from the average of the results indicated by Fig. 1 and corrected for action of gravity on the tissue paper used to determine velocities.

Now having established the approximate relation between velocity and loss of draft by experiment and by theory, and having checked the result from past practice, it is possible to make use of the curve, as follows:

The curves in Fig. 6 are based on the principle that if a 100-ft. stack will give a draft of 0.61 in., with no velocity when the outer air is at the temperature of 70° Fahrenheit, and the stack gas is at the temperature of 500° F., the draft at a velocity of 10 ft. per second will be 0.036 in. less [$R = .00036 \times (10)^2$], as shown by Fig. 5.; at 20 ft. velocity it will be 0.144 in. less.; at 30 ft. velocity it will be 0.324 in. less, and so on. These results, shown in the form of a curve for a 100-ft. stack, give the drafts which may be expected at various velocities. Curves for stacks of other heights are laid off in the same manner.

Since these curves are based on a stack temperature of 500° F., and since this temperature may sometimes be as high as 600° F., it is necessary to consider the effect of this variation. With 600° F. temperature an 88-ft. stack produces about the same draft as a 100-ft. stack with 500° F. temperature. Therefore, if a temperature of 600° F. is expected (from overrating boilers or other causes), and a draft of 1 in. is required, the stack which gives 88 per cent. of this, or 0.88 in., is the one to be taken from the curves of Fig. 6.

To consider now the ability of a stack to carry off gas: The volume of stack gas produced in any power plant is primarily dependent upon the weight of coal burned. It is also dependent upon the temperature of the gas and upon the amount of excess air used in the combustion of the coal. The extreme variation of temperature, with fair practice, is to 600° F., which makes an increase in volume of 10 per cent. The factor of excess air is, however, a large one. One pound of coal when completely burned requires, under very good conditions, 200 cu. ft. of air; under good conditions, 250 cu. ft. of air; under poor conditions, 300 cu. ft. of air.

Thin fires, irregular stoking and excessive drafts are conditions which should not occur in a modern boiler room, and so need not be considered.

Now, 250 cu. ft. of air, after burning 1 lb. of coal and being reduced in temperature to 500° F., makes a volume of about 450 cu. ft. of flue gas; and if the stack velocity is 1 ft. per second, or 3,600 ft. per hour, the area of stack necessary to carry off gas from the combustion of 1 lb. of coal per hour is, under the above conditions, $450 \div 3{,}600 = 0.125$ sq. ft. Therefore, by dividing the area of any stack by 0.125 the number of pounds of coal that the stack will handle at a velocity of 1 ft. per second is obtained; or, multiplying the required weight of coal in pounds per hour by 0.125, gives the necessary stack area for a velocity of 1 ft. per second.

The actual area of a stack, however, is not all effective. There is a layer of gas at the sides which is stationary or nearly so. The thickness of this probably varies with the velocity, but Kent assumes it as 2 in. thick for all stacks, and following this principle as one of long established use, a 4-ft. stack would have an effective area of 10.44 sq. ft., and at 1 ft. per second velocity would carry off gas from 84 lb. of coal per hour. At 5 ft. per second velocity it would handle 418 lb., and at 10 ft. per second, 836 lb.

By applying this method to various sizes of stacks, the table under Fig. 6 was prepared, and with the combined curve and table it is possible to give the necessary height and diameter of a stack which will maintain a given draft at the base while carrying off the gas produced by the combustion of a given weight of coal per hour.

The object of maintaining draft at the base of a stack is to overcome the resistances of flue and boiler and to produce sufficient vacuum over the fire to give the necessary rate of combustion. The relation of vacuum over fire, or furnace draft, to the number of pounds of coal burned per square foot of grate per hour is a subject which is hardly within the scope of this article. It is affected largely by the care with which the firing is done and, with the coking coals of the Eastern States, depends on how often and how well the lumps of coke are broken up. It is also affected by the class of fuel, but as anthracite is no longer a commercial steam fuel, soft coal need only be considered. Owing to the wide use of stokers, soft coal slack will soon be about as high priced as run of mine coal. In fact, at the prevailing prices, run of mine coal is by far the better fuel commercially in almost all parts of the country; hence slack coal need not be considered under ordinary conditions, and with

average firemen the following rates may be expected for run of mine coal:

Eastern Coal.		Western Coal.	
Draft.	Coal per sq. ft. of grate.	Draft.	Coal per sq. ft. of grate.
0.20 in.	14 lb.	0.20 in.	18 lb.
0.30 "	19 "	0.30 "	24 "
0.40 "	24 "	0.40 "	30 "
0.50 "	28 "	0.50 "	35 "

These figures may be greatly exceeded under favorable conditions.

Stirling gives a curve showing a rate of 50 lb. coal per square foot of grate, with a draft of 0.40 in., and Whitham records a test where 0.30 in. gave a rate of 32 lb. of coal per square foot, but such results could not be expected in every day practice. In fact, dirty or uneven fires, holes or large lumps of coke may easily reduce the figures in the table 50 per cent. In one plant examined by the writer a draft of 0.70 in. only produced a rate of 20 lb, with good Pennsylvania coal mixed with 40 per cent. of yard screenings, because the firemen spoiled the fires by excessive slashing and did not keep the surface well broken up and even.

The resistances of the various types of boilers have been found to be, roughly, as follows, when delivering rated capacity under ordinary conditions of operation:

Resistances of Boilers.	
Babcock & Wilcox, double deck	0.40 in.
Babcock & Wilcox, standard	0.30 "
Stirling	0.20 "
Heine	0.20 "
Return tubular	0.10 "
Upright	0.00 "

(For 25 per cent. over-rate multiplied by 1.5.)

The resistance of each 100 ft. of straight flue may be considered as 0.05 in., and the resistance of each bend as 0.05 in. These values are, of course, dependent upon the size of the flue and the velocity of the gas in it, which should be about 20 ft. per second under ordinary conditions. This velocity gives very large flue areas, compared with past practice, but it is nearly always cheaper to build large flues than high stacks.

To take a concrete example: Assume a battery of Babcock & Wilcox boilers, which are to run at rating. They are connected to a horizontal flue 100 ft. long, which makes one bend before entering the stack. The uptake from each boiler enters the main flue at right angles. The boilers are to furnish 3,000 h.-p. by burning a fair grade of Eastern coking coal, giving 1 h.-p. for each 4 lb. of coal burned, and hence 12,000 lb. of coal are to be burned per hour. Assume the total grate area of all boilers to be 500 sq. ft., then 24 lb. of coal must be burned on each square foot per hour, and from the table of rates of combustion it is seen that 0.40 in. of draft must be given over the fire.

Now, there will be needed 0.4 in. draft for the furnace, 0.3 in. to overcome boiler resistance, 0.05 in. to overcome the bend at each uptake, 0.05 in. for the 100 ft. of main flue, and 0.05 in. for the bend in the main flue. This gives a total of 0.85 in. needed at the stack base.

Referring to the vertical division marked "draft" on Fig. 6 and selecting the point corresponding to 0.85 in., it is seen that a horizontal line, drawn from this point, crosses the curve marked "150-ft. stack" directly above the velocity of 15 ft., meaning that a 150-ft. stack will give 0.85 in. draft when the velocity is 15 ft. per second. Extending the horizontal line to the curve of the 175-ft stack, it is seen that the velocity nearest to being below the intersection is 25 ft., meaning a 175-ft. stack will maintain 0.85 in. of draft and a velocity of 25 ft. per second. In the same way a 200-ft. stack is shown to give 0.85 in. of draft and 35 ft. velocity.

Now referring to the table of Fig. 6, it is seen that if 12,000 lb. of coal are to be burned with a stack velocity at 15 ft., or that given by a 150-ft. stack maintaining 0.85 in. of draft, the stack must be nearly 12 ft. in diameter (which diameter will handle 12,800 lb. of coal at this velocity). But if the stack is 175 ft. high and the velocity is 25 ft., the diameter need only be 9 ft., or if the stack is 200 ft. high it need be only 8 ft. in diameter.

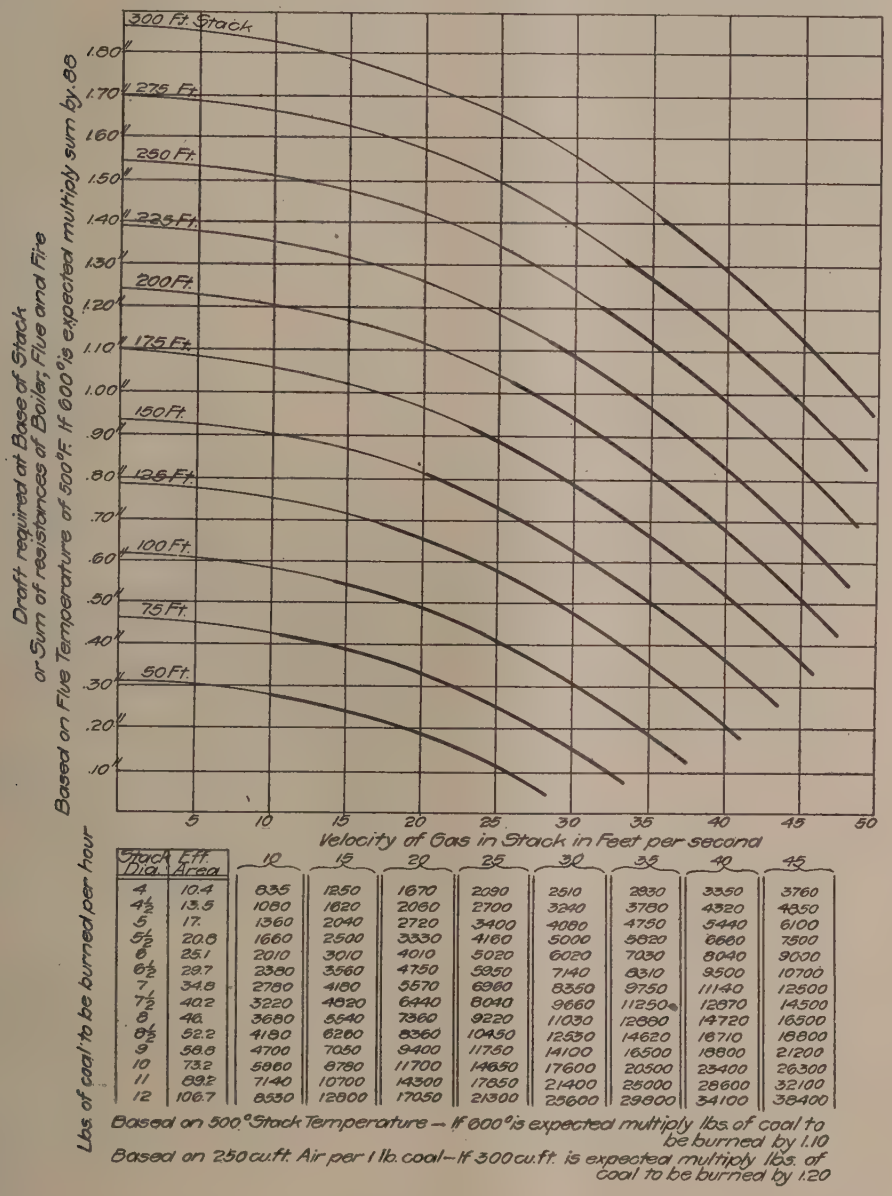
The horizontal line showing 0.85 in. of draft crosses the curve of the 150-ft. stack outside of the heavily marked part of the curve, and this indicates a velocity too low to be economical. Except under extraordinary conditions, the 175-ft. by 9-ft. stack will be cheaper than and preferable to the 150 x 12-ft. It will undoubtedly contain less brick and will be less liable to the effect of outside conditions.

Suppose that it is desired to get the same horse-power by reducing the number of boilers or by over-rating the boilers 25 per cent., and say that the grate area is proportionately larger and the fireman careless. Then it may be expected that the flue temperature will go up to 600° F. and that uneven fires will bring up the volume of air per pound of coal to 300 cu. ft. Then if the total grate area again is 500 sq. ft. the sum of resistances will be 0.30" × 1½ + 0.05 + 0.05 + 0.05 = 1.00 in.; but as the temperature is 600° F., it is necessary to figure on only 88 per cent. of this, or 0.88 in.

The coal actually to be burned will be 12,000 lb. per hour, but since temperature is 600° F., its equivalent, or 12,000 × 1.10 = 13,200 lb., must be allowed for. Also, since excess air is to be increased, the equivalent of coal properly burned is 13,200 × 1.20 = 15,800 lb.

The table shows that a 10-ft. stack handles at 25 ft. velocity 14,650 lb. of coal, and at 30 ft. velocity 17,500 lb. of coal; and as the horizontal line of 0.88 in. of draft crosses the 175-ft stack curve at about 27 ft. velocity, it may be expected that a 175 x 10-ft. stack will handle 16,100 lb. of coal, which is just above the required amount of 15,800 lb., so that the necessary stack is 175 x 10 ft.

To consider another problem which often arises, namely, that of determining the height of stack to be added in case of an increase of horse-power in an old boiler plant: Assume that the foregoing plant has a stack 175 x 9 ft. and that it becomes necessary to add 1,000 boiler horse-power to the plant, the new boilers to be similar to the old ones in every way, and led into the stack by a separate connection or by a properly enlarged breeching. Then the amount of draft at the stack base will not have to be increased, but the velocity of the stack gas must be increased to take care of the extra 4,000 lb. of coal burned per hour.


300 Ft. Stack
275 Ft.
250 Ft.
225 Ft.
200 Ft.
175 Ft.
150 Ft.
125 Ft.
100 Ft.
75 Ft.
50 Ft.
Draft required at Base of Stack or Sum of resistances of Boiler, Flue and Fire
Based on Flue Temperature of 500°F. If 600° is expected multiply sum by .88
Velocity of Gas in Stack in Feet per second
Lbs. of coal to be burned per hour

Stack Dia.	Eff. Area	10	15	20	25	30	35	40	45
4	10.4	835	1250	1670	2090	2510	2930	3350	3760
4½	13.5	1080	1620	2060	2700	3240	3780	4320	4850
5	17.	1360	2040	2720	3400	4080	4750	5440	6100
5½	20.8	1660	2500	3330	4160	5000	5820	6660	7500
6	25.1	2010	3010	4010	5020	6020	7030	8040	9000
6½	29.7	2380	3560	4750	5950	7140	8310	9500	10700
7	34.8	2780	4180	5570	6960	8350	9750	11140	12500
7½	40.2	3220	4820	6440	8040	9660	11250	12870	14500
8	46.	3680	5540	7360	9220	11030	12890	14720	16500
8½	52.2	4180	6260	8360	10450	12530	14620	16710	18800
9	58.8	4700	7050	9400	11750	14100	16500	18800	21200
10	73.2	5860	8780	11700	14650	17600	20500	23400	26300
11	89.2	7140	10700	14300	17850	21400	25000	28600	32100
12	106.7	8530	12800	17050	21300	25600	29800	34100	38400

Based on 500°Stack Temperature — If 600° is expected multiply lbs. of coal to be burned by 1.10

Based on 250 cu.ft. Air per 1 lb. coal — If 300 cu.ft. is expected multiply lbs. of coal to be burned by 1.20

Chart Giving Dimensions of Chimneys for Different Services.

Referring to the table of pounds of coal burned, Fig. 6, it is seen that if 16,000 lb. are to be burned and the stack is 9 ft. in diameter, the velocity will have to be nearly 35 ft. per second, because this velocity in a 9-ft. stack carries off gas from 16,500 lb. of coal per hour. A vertical line drawn from the point showing 35 ft. velocity intersects the horizontal line of 0.85 in. draft just above the curve for the 200-ft. stack, or where the curve for a 205-ft. stack would be; or, in other words, it is necessary to "top out" the old stack 30 ft. This will undoubtedly be cheaper than building a new stack and foundation for the new boilers.

The proper areas for breechings which will give a velocity of 20 ft. per second may be obtained from the column showing effective areas corresponding to the various weights of coal burned per hour in the column under 20-ft. velocity.

Now by comparing the figures given in the first of the foregoing examples and Kent's table of chimney capacity, it is seen that in Kent's table a stack 175 x 8½ ft. will serve 2,300 h.-p. or burn about 12,000 lb. of coal. This is a smaller stack than is indicated in the example, but if the draft over the fire is set at 0.30 in. instead of 0.40 in., and the draft at the stack base reduced to 0.75 in., the table in Fig. 6 shows that about 12000 lb. can be handled by a 175 x 8-ft. stack. If in the same example tubular boilers are used instead of Babcock & Wilcox, and the necessary draft reduced to 0.55 in., the stack may be reduced to 133 ft. in height, if the diameter is kept at 9 ft.; or if the dimensions are kept at 175 x 8½ ft it will burn 16,500 lb. of coal.

Referring again to the formula $H = (0.3 \text{ h.-p.}/E)^2$, with regard to the velocity it gives and the drafts exerted at stack bases, it is evident that, since the height is made a function of horse-power divided by area, the velocity for any height is constant. These velocities, drafts and percentages of total drafts are shown below in a table:

Height.	Velocity.	Effective draft at base.	Per cent. of total draft exerted.
300 ft.	37 ft.	1.38 in.	75 per cent.
250 "	33 "	1.17 "	75 "
200 "	30 "	.93 "	76 "
150 "	25 "	.72 "	78 "
100 "	20 "	.48 "	79 "

From this it is easily seen why stacks designed from this formula give good results if the correct height is chosen. It also accounts for the common saying that for the taller stacks Kent's formula gives excessive diameters, because it is seldom that the draft at the base of a stack has to be over 1.00 in., and consequently there would be 0.17 to 0.38 in. wasted which could have been applied to increasing the velocity and decreasing the diameter.

To return to the questions in the opening paragraph: Reducing a 150 x 6-ft. stack, burning 5,000 lb. of coal, to a diameter of 5 ft. will decrease the draft 0.30 in. Reducing the height to 100 ft. will also decrease the draft 0.30 in. If we add 500 h.-p. to the plant, without increasing the grate area under each boiler, the stack must be increased to 200 ft. in height. Introducing forced draft will allow the furnace draft to be applied to increasing velocity; if this be 0.40 in. it will enable the stack to carry 1,600 h.-p., or burn 8,000 lb. coal per hour.

This article refers in a number of places to the well-known chimney formula originated by Prof. William Kent, dean of the College of Applied Science at Syracuse University. Proofs of the article were accordingly submitted to him and he has furnished the following comments:

"There is one statement by the author that I do not think quite fair to my formula. He says, 'It is manifest that if it gave a satisfactory stack for a battery of Babcock & Wilcox boilers, it would give for a battery of Manning upright boilers a structure vastly in excess of the requirements.' I tried to make it clear in my book that the formula was to be used after the height had been assumed, see pages 734-6.

"I have looked into Mr. Kingsley's paper to see how he would solve a practical problem of burning 12,000 lb. of coal per hour under B. & W. boilers. He says that for a 175-ft. stack, the diameter need only be 9 ft. My table shows that for a 175-ft. stack, 102 in. diameter will give 11,500 lb. of coal per hour, and if, as is generally believed, my table errs on the safe side, then 102 in. diameter may be considered enough for 12,000 lb. of coal, instead of 108 in. that Mr. Kingsley's formula would give. Mr. Kingsley, however, states that if the chimney is 200 ft. high, it need only be 8 ft. in diameter. My figure for a chimney of that size is 2,167 × 5, or only 10,835 lb. of coal, so there is a considerable lack of harmony between Mr. Kingsley's method and my own. For a 175-ft. stack he would give a larger diameter than I would and for a 250-ft. stack he would give a smaller diameter. In the absence of experimental proof I should prefer to use my method.

"Mr. Kingsley's method of proportioning a stack seems to be a rather roundabout one, involving a knowledge of what drafts are required to burn different kinds of coal at different rates of combustion, what velocities are needed in chimneys to carry off the gases, and what resistances to draft there are in different kinds of boilers and in flues and bends. Many of these data are imperfectly known, and one of them, the resistance on the bed of coal on the grate may vary from minute to minute through a wide range. It would seem to be just as well to have a purely empirical method of selecting the height of the chimney; that is, to find out from boiler manufacturers or others what height of chimney they have found suitable for different rates of combustion with different kinds of coal, and then, using that height, take the diameter from my table. In the more than twenty years that my table has been before the public, I have never heard objection to it on the ground that it gave too small an area of chimney. It has often been objected to for giving a chimney larger than was necessary, but that is a good fault, and I have never had sufficient evidence of the fault being large enough to warrant the figures or the formula being changed."

The Mississippi River Bridge, at St. Louis, of the Illinois Traction System, commonly known as the McKinley syndicate, was started on Dec. 8 by launching the foundation caisson for one of the channel piers on the Illinois side of the river. The bridge will span the river from the town of Venice, just north of East St. Louis, on the Illinois side, to the northern part of St. Louis, on the Missouri side, the location being about 8 miles south of the Merchants Bridge. The structure will consist of three channel spans, the middle one 523 ft. long, and the other two 521 ft. long, with three short approach truss spans on the Missouri side and two on the Illinois side. The piers will be of concrete with stone facing and granite coping. Those for the main spans will be carried down 60 to 65 ft. to bed rock, while the approach piers will rest on piles. The bridge has been designed for two 120-ton electric locomotives, and will be heavier than either of the other St. Louis bridges. In general outlines it is similar to the Merchants Bridge, but differs in details. The wagon ways will probably be located in the center at first, but as traffic increases will be placed outside the trusses on cantilever supports. Mr. Ralph Modjeski, Chicago, as consulting engineer for the St. Louis Electric Bridge Co., designed the structure.

Mechanical Plant of the Brooklyn Institute Building.—II.

Ventilation.—The building of the Brooklyn Institute of Arts and Sciences is devoted to a variety of museum and educational purposes, including extensive museums of natural history, archaeology, anthropology and ethnology, a large gallery and sculpture hall, an auditorium seating 1,500 persons and laboratories, studies and work-rooms, all of which are largely public in character, so that it is desirable to provide for thorough ventilation. Mechanical ventilation had been provided for in the original west wing of the building, including both fresh air supply and exhaust ventilation in capacity sufficient to clear the air in all departments when crowded with visitors, and this provision has also been made a feature of the second and third sections of the building, recently completed. Ventilation has been provided for in practically all rooms above the cellar, in capacities sufficient for maximum normal occupancy while in the auditorium special provisions were made for ventilation along the lines of the best practice in theatre and lecture hall ventilation.

The ventilation systems were, owing chiefly to the construction of the building in sections at different times, laid out independently for each section. Each division requires, on account of the size of the building, an equipment of large capacity and extensive duct systems for air distribution and gathering systems, the central section having an additional system independent of that for general ventilation which is intended to serve the auditorium only. There is to be noticed a lack of uniformity in capacities between the ventilating equipment in the different sections which is due largely to the varying purposes for which the different portions of the building are used. The systems vary in capacities from 30,000 to over 50,000 cu. ft. per minute of fresh air supply and from 26,000 to 50,000 cu. ft. of exhaust, the capacity of supply generally exceeding that of exhaust ventilation to create a plenum within the building. The fresh air supplies are all cleansed by cheese-cloth filters of the usual construction and the systems are fitted with tempering coils and moistening pans for the control of the temperature and the humidity of the air delivery.

The ventilation is planned throughout the entire building for downward circulation of air, the fresh air supply being admitted through registers in the upper side walls near the ceiling line, except in special cases where the outlets are located in the ceiling, while the exhaust register openings are almost all located at or near the floor level; in toilets and where heat removal was deemed of importance, there are top exhaust openings. Thorough distribution of the supply is provided by well-studied arrangements of registers in all rooms, which, in their relation to the exhaust registers, direct currents of air into all corners. In all of the larger rooms there are two or more sets of supply and exhaust registers, there being in some of the larger museum and laboratory rooms, of a maximum size of 38 x 108 ft. in plan, as many as six and eight registers each for supply and exhaust. In the smaller rooms such as used for offices and auxiliary work rooms which range in size from 12 x 20 ft. to 15 x 40 ft., there are uniformly two each for supply and exhaust, but the locations chosen are so separated as to render the circulation general and positive. Exceptions to this are to be noted only in the auditorium, where a special system has been installed and in the toilet rooms, which, of course, have exhaust ventilation connections only, with fresh air supply connections omitted other than in the form of louvres in the panels of the doors connecting with the

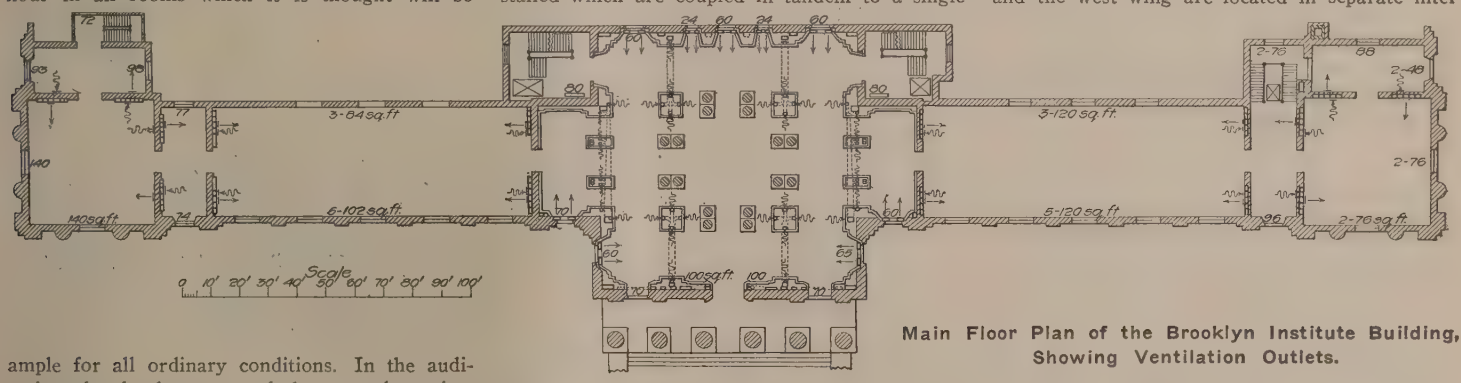
outside or the corridors through which air may enter to replace that exhausted. The general arrangement of register openings was affected somewhat by the desire to confine the duct risers connecting with them to certain rows of inside partition walls in which they may be concealed with the least amount of interference with the interior arrangement and the daylight lighting from side windows. As may be noticed from the general first floor plan of the building, in the original west wing, all of the duct risers are arranged in the cross-wise partition walls with none in the side walls, but in the new easterly wing ducts have been placed in the side walls to a limited extent where found desirable for better distribution connections. In the central section of the building the duct risers are very generally distributed, being carried up for the greater part in furred enclosures which were provided for the interior decorative construction.

As there were no definite ventilation requirements imposed in the various parts of the building, as the conditions of occupancy are in all portions except the auditorium irregular and indefinite, the design of the ventilating equipments was merely based on the desirable ventilation rates of from three to four changes of air per hour in all rooms which it is thought will be ample for all ordinary conditions. In the auditorium in the basement of the central portion, however, the conditions were more definite, ventilation being required at the rate of 30 cu. ft. per person for the average audience expected in attendance. The sizes and arrangements of the fan equipment were decided by the construction of the different sections of the building at detached intervals, a single unit each for both fresh air supply and exhaust being installed in each section of sufficient capacity to handle that general division of the building, while basement or cellar locations for the fresh air supply blowers, with convenient filtered air intakes, were chosen with exhaust fans in the attic. In the original west wing, the fresh air blower was located in the west end of the basement, the greater part of which floor is utilized for museum purposes, and the distribution duct line concealed in furred ceiling construction, but in the central portion and the new east wing, the sharp descent in the grade at that portion of the site facilitated the provision of sub-floors which are used solely for the mechanical equipment and thus without waste of valuable space above ground level. In the two latter cases the distribution duct lines are carried on the cellar ceiling exposed, the trunk duct and branches being covered with asbestos block and canvas. For the fresh air supply, fans of the centrifugal type were selected in all cases, while for exhaust, disc fans were installed in attic locations where convenient to the gathering duct systems and roof discharge outlets, excepting in the case of the auditorium system, where owing to concentrated space ventilated it was thought desirable to locate this fan in the basement alongside of that for fresh air supply. The fans are all motor driven with the exception of that for fresh air supply in the old west wing, in which case the direct connected steam engine drive which was originally installed, has been retained. In the case of the auditorium systems where both fresh air supply and exhaust fans are both located in the basement, they have been arranged side by side with a tandem drive from a single large motor.

The principal details of the fan equipments, their drives and tempering coils are presented in an accompanying table. The fans of the centrifugal type are all three-quarter housed steel plate blowers supplied with the tempering coils by the B. F. Sturtevant Co., and are for the greater part installed with top horizontal discharge casings. They have single direct connected drives with the exception of the systems supplying fresh air to the central portion of the building where, on account of the large volume of supply required, and also of lack of space for a very large unit, two smaller units were installed which are coupled in tandem to a single motor drive and have connections in multiple for both intake and delivery. The motors installed are all C. & C. direct-current, slow-speed motors which are operated on separate power circuits from the local power plant supply, their control being in the local distributing switchboards in each section of the building; they are fitted with Cutler-Hammer starting boxes fitted with both overload and underload release to permit of their being operated with a minimum of attention. The fresh air blower in the west wing is driven by a 10x12-in. Sturtevant horizontal automatic steam engine which, as stated in the preceding article, receives steam from the high pressure system from the power plant and exhausts locally into the low pressure heating main which centers in that wing.

The fresh air fans have short direct intake connections made in the most convenient manner to the outside air, at low levels, the intake air being cleansed by cheesecloth filters. The west wing fan intake is through a grilled window opening in the west end of the basement, that for the east wing fan through a window and area way at the front, while that for the two fans in the central portion is through a masonry enclosure outside of the building wall which is connected with a 3x24-ft. galvanized iron enclosure that is carried up to a hooded opening about 12 ft. above grade level at the rear of the building. Each of the intakes is fitted with a filter of the usual zigzag frame construction with cheese-cloth as the filtering medium, the areas of which cloth are proportioned in relation to the maximum fan capacities as to secure low rates of filtering velocities, which range from 50 to 70 ft. per minute in the various filters. The higher filter velocity of approximately 70 ft. per minute, is to be found in the filter for the central portion which supplies the two fans, one for general ventilation and the other that for the auditorium; as the auditorium is to be used only occasionally and presumably at such times as the remainder of the building will be closed, an area of filtering medium was chosen which, while causing a rather high rate of flow if for both systems should happen to be operated to maximum capacity simultaneously, will in all probability be rarely if ever operated under this condition of maximum capacity. The filters are of the usual construction consisting of light wooden frames carrying wire netting over which the cloth is tightly stretched, which are mounted in light galvanized iron framework with fittings for clamping them in place air-tight; duplicate filter screens are provided to enable replacements of those in service for cleaning. The filters for the central portion and the west wing are located in separate filter chambers of ample size which open directly to the outside and from which the fans draw through short duct connections; in the west wing system the fan is itself located directly within the filter chamber enclosure, thus avoiding the intake duct connections. In the two former cases, however, the tempering coils are conveniently located in the intake duct connections between the filter chamber and the fan intake, permitting direct connections from the fan delivery openings to the delivery duct lines, while in the west wing system it was necessary to provide space in the fan delivery ducts for the coils. The tempering coils are of the Sturtevant mitre pattern of 1-in. pipes, which are installed in two row sections throughout in numbers of sections and in capacities of surface indicated in the accompanying table. The steam supply and vapor and condensation return connections to these coils were referred to in the description of the heating system of the building in the preceding article. These coils are like the direct radiation fitted for automatic temperature regulation, each system having two control thermostats, one in the outside air operating the supply and return valves of the two outside pipe sections only when the exterior temperature drops below freezing and the other inserted within the delivery ducts for the close adjustment of the air delivery to the temperature required.

The duct systems have been liberally proportioned for low velocities of flow, rates of from 1,800 to 2,000 ft. per minute being prevalent in trunk duct lines which are decreased to lower values in the branches and finally to velocities of outflow not to exceed 350 ft. per minute through the outlet registers. The duct systems in the side wings are comparatively simple, consisting of a single longitudinal trunk duct through the

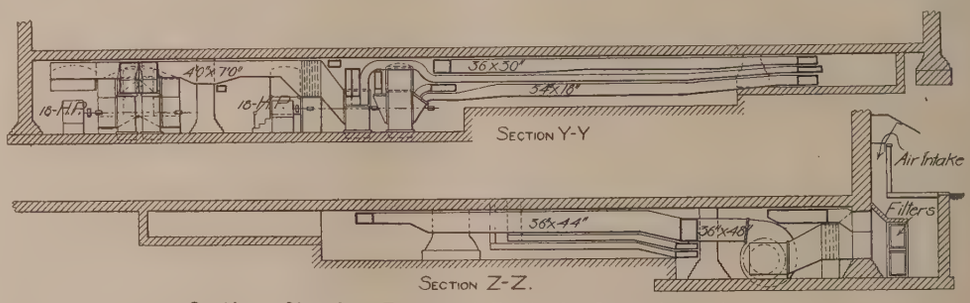


Main Floor Plan of the Brooklyn Institute Building, Showing Ventilation Outlets.

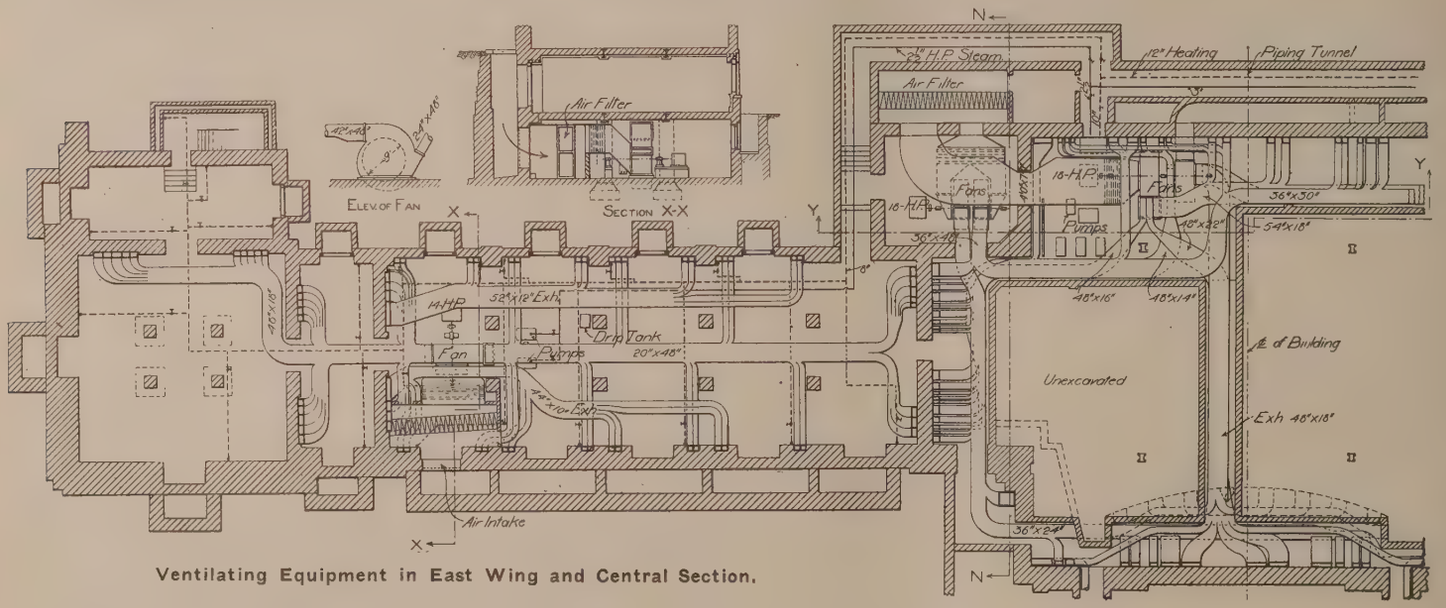
DETAILS OF FAN EQUIPMENT FOR VENTILATION OF BROOKLYN INSTITUTE BUILDING.

Section Served	Size of Fan Wheel. Diam., ft.	Width, ft.	Size of Motor, h.p.	Speed, r.p.m.	Capacity, cu. ft per min. Supply.	Exhaust.	Tempering Coils. No. of Sections.	H't'g Surface, sq. ft.
West wing	8	4	(10x12-in. engine)	140	31,600	.....	8	1,845
West wing	5	disc	10	140	.....	30,700	..	.....
Central portion	2-7	3	18	165	54,100	.....	10	2,000
Central portion	7	disc	10	225	.....	49,700	..	.....
Auditorium	8	4	18	140	31,500	.....	7	1,800
Auditorium	7	4		140	.....	26,000	..	.....
East wing	9	4½	14	130	43,000	.....	7	2,000
East wing	6	disc	15	275	.....	38,000	..	.....

center of the cellar or basement with branches to the banks of partition risers to the upper floors; these risers of which there are one to practically every floor in each group, are made uniformly 12 in. in depth and vary in width, provisions being made for them in the furred wall construction in the various cross-wise partitions of the side wings. In the central portion of the building the arrangement differs somewhat, trunk duct lines extending from each of the two fans for general ventilation around the four side walls of the basement with branches to the duct risers that are carried up in the furred side wall faces. The general arrangement of the supply duct work, as well as also that for exhaust ventilation, are well shown in the accompanying drawings. The exhaust ducts and connections follow in fact the general arrangement of those for fresh air supply, duct risers being carried up also in the cross-wise partitions of the wings and in the furred side wall construction of the central portion to gathering duct lines in the attic which connect with the disc exhaust fans. The supply and exhaust duct risers are in fact alternated, the exhaust connections to the upper floors occupying spaces directly above those for supply to the lower floors with consequent economy of space. It was necessary to arrange the gathering duct lines in the side wing attics in two divided lines on either side owing to the provision of the extensive skylights in the middle portion of the roofs for the art gallery lighting. These lines and



Sections Showing Fans and Ducts in Central Section.



Ventilating Equipment in East Wing and Central Section.

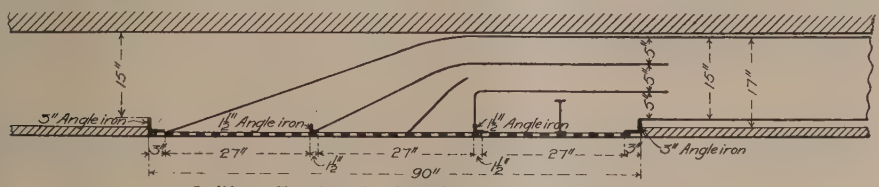
the branch lines from the end sections of the wings lead in both cases to exhaust chambers near the central portion of the wing from which disc exhaust fans discharge through roof outlets. In the case of the central portion of the building the exhaust fan is located in the attic space in the rear of the base of the dome and draws through two gathering ducts that extend on all four sides of this section. The exhaust duct lines are similarly designed for low velocities of flow, averaging but little over 1,000 ft. per minute, and the ducts are without insulation covering throughout.

The exhaust fans are Howard & Morse disc exhaust fans of the sizes and capacities indicated in the table and are of the vertical type in the central portion and east wing systems while that for the original west wing was installed horizontally. They are all direct connected to C. & C. direct current electric motors, special vertical shaft motors being used for the two former units. These motors all have separate feeder circuits from the distributing switchboard in the basement to permit of being shut down directly from the latter board and requiring attention only on starting. These fans all draw from spacious exhaust chambers into which the branch gathering ducts connect and deliver direct to the outside through exhaust hoods in the roof with openings of the full size of the fan in each case. These roof discharge hoods are of heavy copper construction, well braced and are specially designed to protect the fan and motor from the weather, while not restricting the outflow.

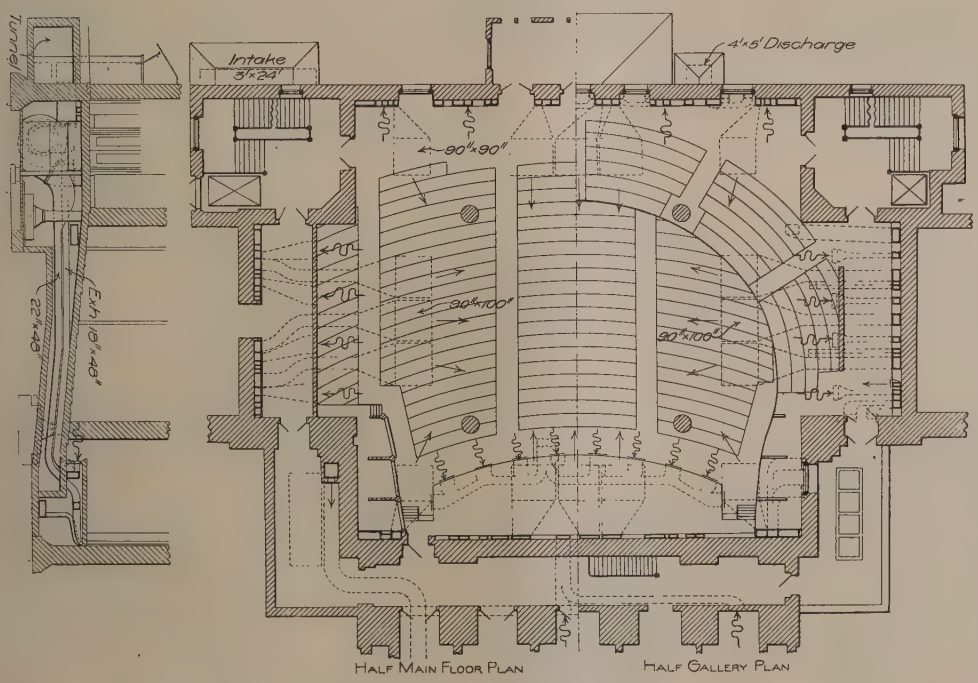
Auditorium System.—The ventilation of the auditorium has, as above stated, been provided for in accordance with approved public hall practice, air being circulated on the downward system and capacity provided for approximately 30 cu. ft. per minute per person for an average audience of a little over two-thirds of the seating capacity. The seating capacity of the hall is about 1,500, of which there are about 1,000 seats on the main floor arranged in five rows and about 500 seats in six groups in a balcony that surrounds the hall on three sides, with in all, 12 boxes, six on either level. The main floor slopes downward to the front towards the rostrum, to a point 4 ft. below the level of the rostrum, the latter and the rear floor and corridors being on the level of the basement floor of the building. The rostrum is 16 ft. in width by 58 ft. long, and the hall itself 104 ft. in maximum length, parallel to the rostrum, and 86 ft. in width, with a ceiling height of 26 ft., giving thus a total cubical content of approximately 235,000 cu. ft. With this arrangement of the gallery and the seating, it was thought preferable to admit the fresh air supply through distributed points in the ceiling, around the outer edges, and to withdraw the foul air in the main hall through openings under the rostrum, and in part through ceiling registers at the extreme rear edges and sides of the room both under and over the gallery. In this way it was felt that by designing for low velocities of register flow the most effective distribution could be secured without noticeable draft.

The fan equipments both fresh air supply and exhaust are, as above stated, located in the sub-basement space under the rear of the main floor where convenient for short connections to the outside air. The fans are both Sturtevant centrifugal blowers, of sizes and capacities indicated in the accompanying table, and as they are intended only for simultaneous operation when the hall is occupied, they were located side by side and fitted with tandem drive from a single large motor. They both have three-quarter housed casings, the exhaust blower with top horizontal discharge outlets, while the fresh air fan, being connected to the same drive, is necessarily of the 45-deg. upward discharge type. The arrangements of duct work connections are shown in the accompanying drawings. The fresh air supply fan has intake in common with that for general ventilation of the central portion, connection from the filter chamber to the fan being made through an intake duct of some length located on the ceiling of the sub-basement above the other supply fan. The exhaust fan has its discharge at the rear of the building through a short stack of height similar to that for the central portion fresh air intake, but for purposes of separation of the intake and discharge, the latter stack, which is 4 x 5 ft. in section, is located some 75 ft. to the west of the intake stack, connection being made to the fan discharge through a 4x4-ft. underground masonry duct immediately outside of the building wall. Fresh air is delivered to the auditorium through 16 ceiling outlet registers which are well distributed over the ceiling area. Twelve of them are registers with 90x100-in. faces which are arranged in groups of three over the center of each of the four sides of the room, while the remaining four registers have 90x90-in. faces and are distributed in the corners of the ceiling. Each of the outlet registers is, as may be noted from the accompanying detail, divided into three sections with distributing baffles in the connection to the ducts in order to spread the outflow evenly over the register face and prevent

any tendency toward draft. The delivery velocities for which this system was designed are exceptionally low, the flow velocities averaging from 1,500 to 1,700 ft. per minute in the trunk duct lines from the blower which are successively decreased in the risers and branches to about 800 ft. per minute at the connections to the registers in which the discharge velocities are reduced to only about 50 ft. per minute, thus precluding the possibility of a draft being felt in any part of the room. These ceiling registers are supplied through three main duct lines from the fan in the basement, one of which extends through passages under either side of the auditorium, and the other through a longitudinal passage under the middle which connects with the cross passage at the rear under the rostrum. From these connections are made with the 16 risers which have been carefully proportioned for equal distribution of flow throughout the system.



Ceiling Fresh Air Supply Outlets in Auditorium.



Auditorium Ventilation System.

The exhaust system of the auditorium which is of capacity slightly less than that for fresh air supply, comprises a total of 34 vent registers which are well distributed throughout the room in relation to the fresh air supply openings for the purpose of creating a thorough and positive circulation. Of these vent registers, there are 10, each with 14x36-in. faces and comprising together about one-third of the total exhaust ventilation capacity of the system, which are located in the front portion of the rostrum. Of the remainder 12 are located in the side and rear walls of the room under the balcony, and the other 12 similarly in the side walls above, all of these registers being located at the ceiling line with the exception of the four at the rear under the balcony; at this portion of the main floor there are doorways leading out into the area way at the rear for purposes of fire exits, and here the register openings were located near the floor line. This arrangement divides the remaining two-thirds of the vent register capacity between the spaces at the rear both above and below the balcony, obtaining thereby the most complete distribution of register openings. Similar low velocities of flow are also provided for in this system, the duct lines being proportioned for maximum velocities of less than 1,500 ft. per minute, while the connections to the registers are proportioned for velocities of inflow of less than 250 ft. per minute. The arrangement of the exhaust duct work is also similar to that for fresh air supply, there being three main trunk lines leading to the fan intake, one through each of the side passages, and the other through the central passage to the rostrum intake openings at the rear. Riser branches are carried up from these to the side and rear wall register connections in arrangement similar to that of the supply system risers, in the outer walls beyond the side corridors, the connections to the registers from the side walls being made through furred ceiling construction over the corridors, 16 in. deep.

The entire ventilating equipment of the institution was designed, as well as also the heating system and power plant, by Mr. Alfred R. Wolff, consulting engineer, New York. Messrs. McKim, Mead & White, New York, are architects for the institute buildings.

THE LIMIT OF SUPERHEATING desirable for Corliss engines was stated in a discussion of papers on superheated steam at the recent New York meeting of the American Society of Mechanical Engineers, to be to a temperature of 500° Fahr. With steam used at higher temperatures, the valves give trouble by warping and binding, due to their unsymmetrical form, and also in certain cases cylinders have been known to change shape sufficiently to give trouble. In engines fitted with slide and gridiron valves, however, no difficulty is experienced except with lubrication and that has in certain cases been overcome by special direct oil connections to the valve seats.

Some Main Line Revisions on the Baltimore & Ohio R. R.

The Baltimore & Ohio R. R. has completed a revision of its main line between Hollafield and Davis, Md. This is a continuation of work that has been going on for the past seven years in straightening the alignment and making a low-grade freight line from Relay to Washington Junction, the former place being its junction point with the Washington branch, and the latter its junction point with the Metropolitan branch; these two branches forming its line via Washington, by which route its through passenger trains are run. That part of the main line between Relay and Mt. Airy follows the Patapsco River, the valley being narrow and crooked, with hills rising from the water's edge and spurs from the hills projecting across the bends at intervals which have made it necessary in the reconstruction to build a number of tunnels and bridges, as the original location followed closely the windings of the stream resulting in heavy grades and curves.

T. A. Shoemaker & Co., contractors, of Philadelphia, began work on the revision of the line between Hollafield and Davis, in September, 1905, and completed it in October, 1907, during which time they excavated 400,000 cu. yd. of material, 70 per cent. of which was solid granite, the balance being sand, loam, and clay; 10,000 cu. yd. of bridge masonry of Pennsylvania sand stone; 5,000 cu. yd. of concrete in foundation; two tunnels having a combined length of 1,523 ft.; three undergrade crossings; reconstructed a highway 5,000 ft. in length, and changed the channel of the river in two places.

The grading averaged 100,000 cu. yd. per mile, 187,000 cu. yd. being in the Alberton cut, which was 2,000 ft. in length and 75 ft. high, and practically all the earth on the entire work was in this cut, between the original surface and a point about 25 ft. above subgrade. All the material was removed with a 70-ton Bucyrus shovel working in lifts from the surface to subgrade, bringing it down full-width, and wasting the top material on tracks on the same grade with the shovel, running around the contours of the hill.

The material in the east end on the south side was found to be a conglomerate, there being pockets of mica sand, red clay, and ledges of limestone alternating, which caused the south slope to break and pockets of this material to slide out at the seams. These slips were removed with the shovel. The balance of the excavation was all granite, which was removed with steam drills, and shovel. Dorsey tunnel, one mile west of Alberton and on the opposite side of the river from the old line, with a length of 1,025 ft. face to face of portals, was bored through solid granite, and only 96 ft. at the west end had to be timbered, on account of seams in the roof. A top heading, 7x10 ft., was first driven from both ends, the balance of the material being taken out in two benches. The amount of excavation in this tunnel was 36,500 cu. yd.

The material in Davis tunnel, one-half mile east of Davis and on the same side of the river as the old line, proved to be a mixture of mica sand, soft granite which disintegrated when exposed to the air, and a hard rock which followed the subgrade closely, at times reaching as high as the wall plate grade 14 ft. above subgrade. The length of this tunnel is 498 ft., face to face of portals, and it had to be timbered for its entire length. Seven segments of 12x12-in. timbers were used for the centers, and plumb posts 3 ft. center to center, also 12x12-in. for two-thirds of its length. Where hard rock was encountered at wall-plate grade the segments rested on wall plates on the rock bench.

The method used in driving this tunnel was

varied from that at the Dorsey tunnel, to the extent that the first heading was excavated from the roof to the wall-plate grade and was taken out full width, the bottom bench being taken out later, the excavation amounting to 17,000 cu. yd. On Oct. 22, 1906, while this tunnel was being lined, 98 ft. of lining having been completed in the east end, a fall occurred 140 ft. from the east end, 30 ft. in diameter almost on the center line, and extending to the original surface of ground 92 ft. above subgrade. This fall was almost plumb and had the appearance of being a shaft, the north side overhanging slightly, having a well-defined rock seam, and from that point south the material was soft granite and mica sand. The contractors began to line the tunnel from the west end and finally got to within 104 ft. of the lining from the east end. They tried several times to get under the fall by using crown bars, and on four different occasions were within a few feet of getting through when the southwest corner would give away and crush their timber centers. They finally gave this method up, and in June, 1907, when the size of the fall had increased to 53 ft. at the roof of the tunnel and 140 ft. at the surface of the ground, they began the use of a Model A Marion shovel and had the fall removed, which consisted of 36,000 cu. yd. and formed a natural slope to original grade. The tunnel was completed Sept. 25, 1907.

The Hotel St. Mark in Oakland.

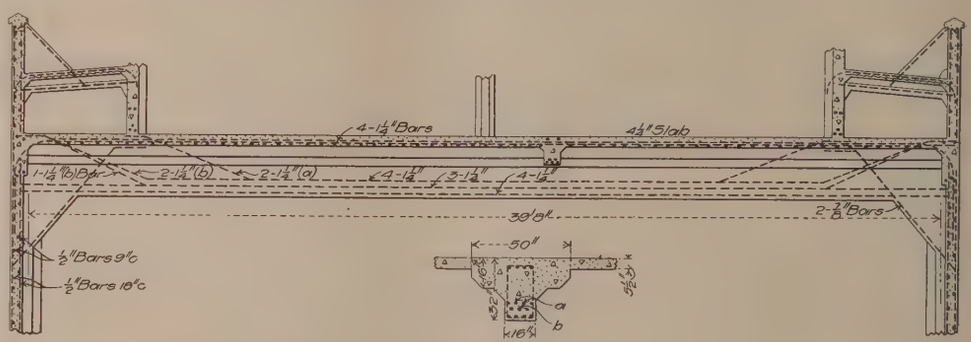
Both tunnels are standard section for double track, and lined with shale and vitrified brick and have sandstone portals. Both were driven by compressed air, the compressor plant being located midway between them, and the air conveyed to them by a 4-in. pipe. The excavation was unclassified.

The old roadbed was entirely abandoned except for one-half mile west of Hollafield, which was raised 5 ft. above the old grade. East of Alberton the new line was built south of the old line through a rock cut, and from Alberton to just east of Davis tunnel the new line runs on the opposite side of the river from the old line, crossing it at this point 12 ft. above the old grade and again crossing to the north side of the old line at the west end of Davis tunnel 8.2 ft. above the old grade, where it continues on this side until Davis is reached, where it connects with the old line.

The revision eliminates 542° 02′ of central angle, and reduces the maximum curve from 12° to 7°, all curves being spiraled. The double-track roadbed is 39 ft. in cuts, and 33 ft. on fills, with 85-lb. rails and stone ballast. The west-bound grade has been reduced from 0.9 to 0.5 per cent. and the east-bound from 0.8 to 0.5, all grades being compensated.

The cost of the improvement has been $750,000 and the work has been done under the direction of Mr. D. D. Carothers, chief engineer; Mr. A. M. Kinsman, engineer of construction, and Mr. J. T. Wilson, assistant engineer.

The Bay Shore Cut-off of the Southern Pacific Railroad in San Francisco was put into service last week. The new line eliminates the disadvantages of the old coast line route in entering the city, and cuts out about five miles of distance, several hundred degrees of curvature and some heavy grades. The ranges of hills to the south of the city are pierced by the new line, necessitating about two miles of tunnels, two miles of trestle and a cut 95 ft. deep, all within 10 miles.



Design of Ninth or Sample-Room Girders.

A Reinforced Concrete Hotel Building in Oakland, Cal.

By Edw. L. Soulé, Assistant Engineer with John B. Leonard, San Francisco.

An interesting example of the use of reinforced concrete is furnished by the Hotel St. Mark, now nearing completion at the corner of Twelfth and Franklin Sts., Oakland, Cal. A view of the finished structural framework, as recently completed, is shown in an accompanying picture. The building consists of two rectangular structures, with a connecting portion, the whole having a shape resembling somewhat that of the letter H. The building rises to a height of 108 ft. above the street level and includes nine floors, a mezzanine floor and a basement. On the outside the concrete skeleton is covered with cement plaster and provided with moulded ornamental trimmings and galvanized iron cornice. The interior finish and partitions conceal the beam and girder construction. The present description will treat of the reinforced concrete work. The main entrance on Twelfth St. leads through the arched lobby to the hotel office in the central portion. The entire Franklin St. portion will be elaborately fitted for a cafe and banquet hall, while the hotel lobby, parlor, organ loft, kitchen and bar occupy the remaining space of the first floor. The upper floors will furnish about 250 rooms equipped for hotel accommodations. The ninth, or sample room floor, will afford commodious display rooms for commercial men.

The plan of the foundation, shown in one of the diagrams, consists of simple, spread and continuous footings, which bring a load of about three and one-half tons per square foot upon a firm clay soil. The lot line footings are carried partly on continuous and partly on cantilever footings. The eccentric moment of the continuous wall footing is overcome by reinforcing the basement wall vertically and by placing tension on the main slab bars of the first floor. Cast-iron plates were used in the majority of the footings to distribute the load carried by the column bars. Lapping bars were also provided, thereby securely anchoring the columns to the foundation and affording sufficient rigidity to move with it.

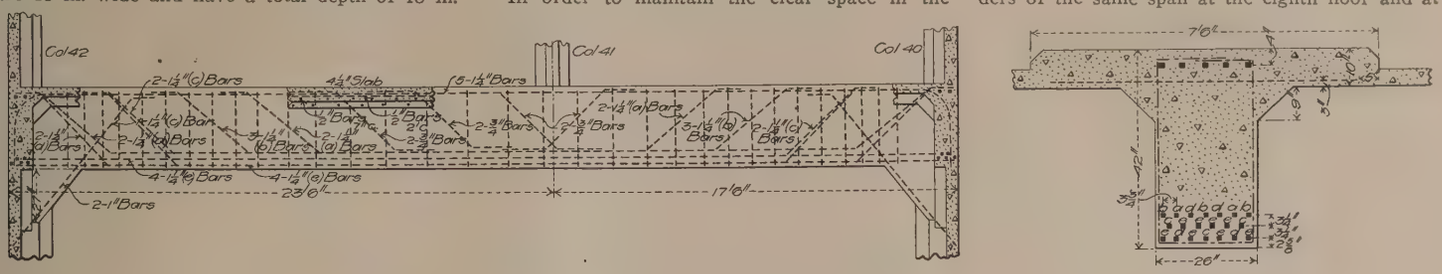
In the basement the columns have an octagonal cross-section. The vertical reinforcement varies according to the load, and is wrapped spirally with soft iron wire. In the majority of cases light reinforcing bars were used, and in some places a double core of eight bars each was provided. The outer bars form the basis for wrapping. The inner core, composed of large-sized bars, was strapped with ¼-in. soft wire at 12-in. intervals. The interior columns above the first floor are square, except two columns between the first and second floors, which were made round for architectural reasons. The corners of all square columns and beams were chamfered by nailing small triangular strips in the molds.

Half plans of the typical and second floors are

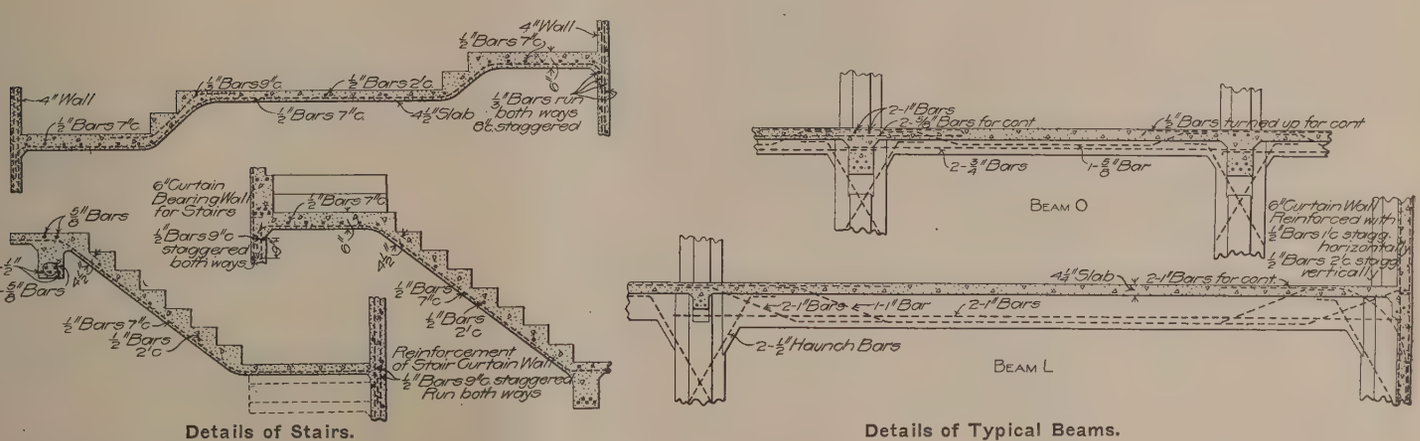
shown in one of the diagrams. In order to afford facility for rapid construction, the design adopted was that of a flat slab type with supporting beams between columns. The transverse beams L between the columns in the interior row are 10 in. wide and have a total depth of 18 in. ing about four-tenths of the main reinforcement in the upper side of the beam at points of support. These continuity bars extend from the quarter point of the clear span to the corresponding quarter point of the adjacent span.

In order to maintain the clear space in the tensional stress. The bars were obtained in special lengths and required no splicing.

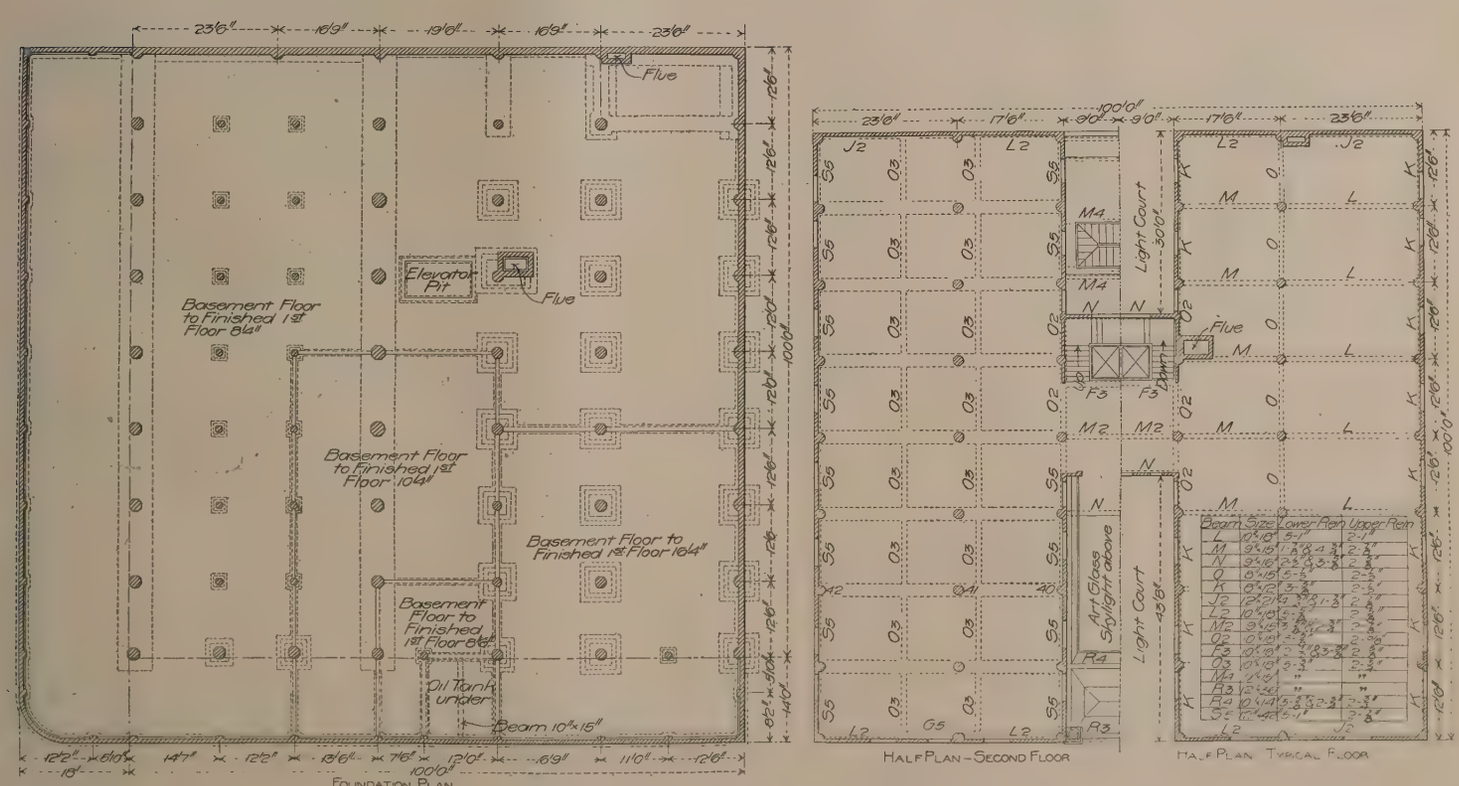
Because of the limitation of size of the second-story girders and to reduce the load on the columns, there were also constructed suitable girders of the same span at the eighth floor and at



Design of Girders over the Banquet Hall.



Details of Stairs. Details of Typical Beams.



Foundation and Floor Plans of the Hotel St. Mark, Oakland, Cal.

They are each reinforced with five 1-in. corrugated bars. The M beams are 9 in. wide and 15 in. deep, reinforced with one ⅞-in. and four ¾-in. corrugated bars. The O longitudinal beams are the same size as the M beams, with five ½-in. reinforcing bars each. In the details of a typical five-bar beam it will be seen that one bar turns up at the quarter point of the clear span, two bars at the eighth point and two bars continue over the supports. Due consideration has been given to the action of continuity by affording about four-tenths of the main reinforcement in the upper side of the beam at points of support. These continuity bars extend from the quarter point of the clear span to the corresponding quarter point of the adjacent span.

In order to maintain the clear space in the banquet hall, it was found necessary to terminate the intermediate columns on girders extending from wall to wall, affording a clear span of 37 ft. 8 in. This was accomplished by means of the tee girders shown in one of the detail diagrams. The architect's plans limited their size to 26 in. width and 32 in. depth to the underside of the slab, on account of the panel depth and finish of the cafe ceiling. Accordingly, the section consisted of five 1¼-in. bars placed in the compression area and twenty 1¼-in. bars to resist the tensional stress. The bars were obtained in special lengths and required no splicing.

Because of the limitation of size of the second-story girders and to reduce the load on the columns, there were also constructed suitable girders of the same span at the eighth floor and at the ninth, or sample room floor. The eighth floor girders are 16 in. wide by 25 in. total depth, and are each reinforced with seven 1¼-in. bars in the lower flange and two 1¼-in. bars in the compression flange. The ninth, or sample room, girders carry the roof loads concentrated on the columns and their own floor loads. One of the diagrams shows the details of sizes and reinforcement.

The first floor was designed for a live load of 120 lb. and the typical floor for a live load of

60 lb. per square foot. Intermediate beams were used on the first floor, thereby reducing the slab to 3½ in. in thickness. The typical floors consist of a 4¼-in. slab. Over the rough slab is laid a ¾-in. finishing coat of cement grout, making a floor with a total depth of 5 in.

A large number of small openings for vents, electric wiring and pipes were required in the floors at various places. These areas were, however, reinforced with additional bars and enclosed by the partition framing.

All walls were built monolithic with the rest of the building and were usually 6 in. thick above the first floor. The reinforcement consisted of one-quarter of one per cent. of the sectional area, one-third of which was placed vertically and the remainder horizontally, all bars being staggered between the outer and inner surface of the concrete. The basement walls serve as retaining walls and are reinforced vertically on the lot line sides. The sidewalk retaining wall varies from 9 in. at the top to 12 in. at the bottom, and is reinforced horizontally between pilasters, which serve as vertical beams.

An interesting portion of the construction is presented in the varied stairways, which illustrate the adaptability of concrete to special details. Attention is directed to the diagram giving plan and detail of typical stairways. The flat soffit allowed of minimum form work and proved an easy type for erection. Near the entrance there is a spiral stairway winding to the basement; also, in the central portion, a circular stairway rises to the orchestra balcony.

For the columns, beams and slabs, the concrete used consisted of one part of cement and six parts of a sand and gravel mixture; for the foundation the proportions were one part cement and seven and one-half parts of aggregates. Golden Gate cement, manufactured at Cement, Cal., about forty miles from the building, was used almost exclusively.

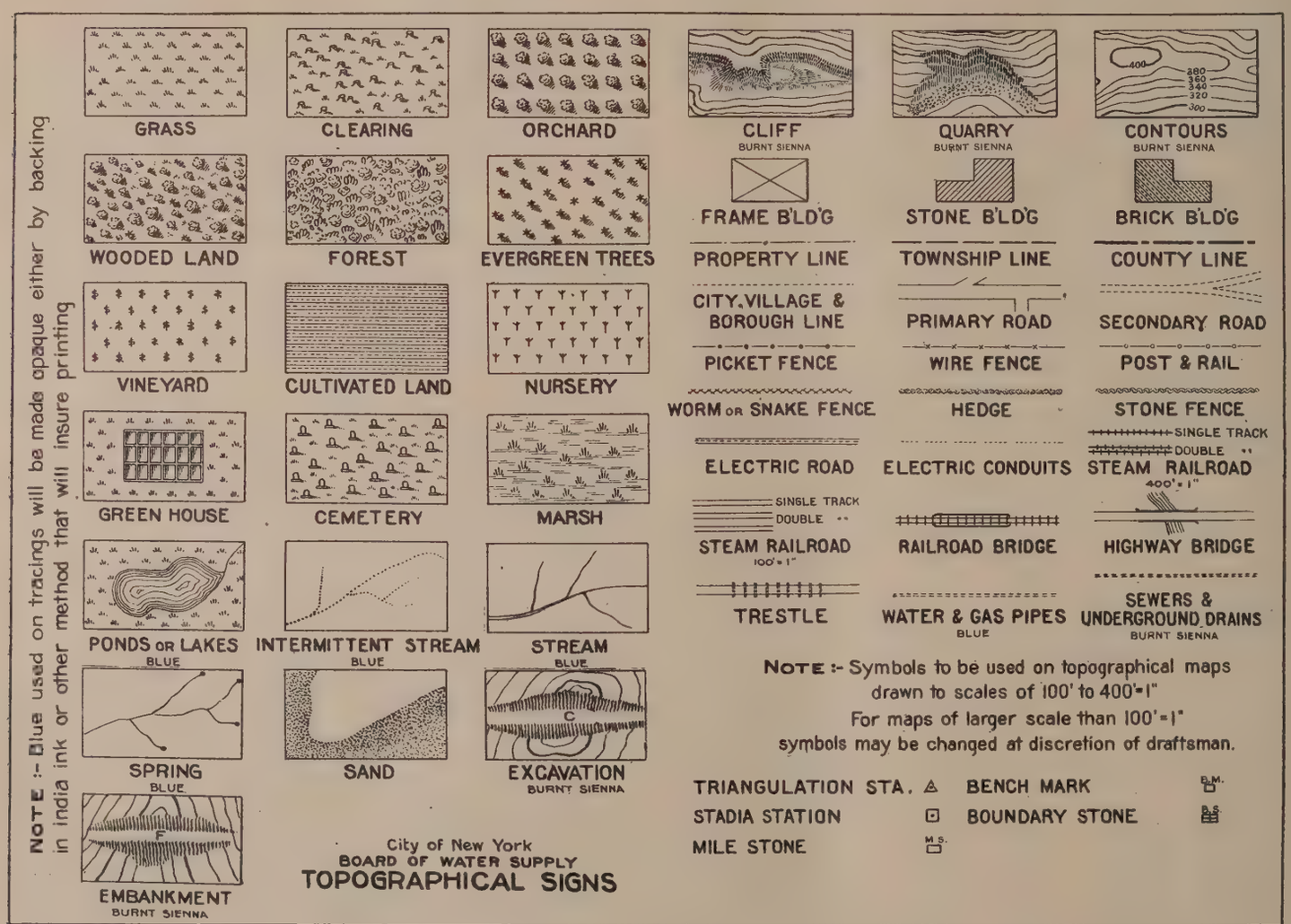
The natural mixture of sand and gravel were of such proportions as to be suitable for direct use. Two adjoining storage bins, one for the cement and one for the sand and gravel, were installed in the basement, near a Smith mixer that discharged the concrete into a hoist. This hoist was situated in the front light well, near the connecting portion of the two rectangular wings, and elevated the concrete to a dumping bin at the required height, from which carts were filled, wheeled over the forms and dumped as required. The concrete was mixed very wet.

Heavy rains considerably impeded the progress of excavation during the month of January, 1907. However, by March 1, the concreting of the foundation was commenced, and after the second story was passed an average progress of a floor per week was made, the fourth story being poured in five and one-half days. Ninety-nine yards were deposited in one day during the fabrication of the second story girders, and 65 yd. were recorded in four hours during the pouring of the eighth floor girders. By July 3 the building was completed structurally except the ceiling beams of the entrance lobby, which were omitted because of the hoist. The concrete was practically completed in 98 working days. On the work there were employed an average of 90 workmen, exclusive of plumbers and electricians, for about 130 days. About 2,520 yd. of concrete and 290 tons of reinforcing bars were used in the structural framework. All reinforcements were made with Johnson corrugated bars, proportioned for a unit stress of 16,000 lb. per square inch. In general, concrete is figured to carry a compressive stress of 600 lb. per square inch.

The architect of the building is Mr. Benjamin G. McDougall, who placed all the structural details of the building in the hands of Mr. John B. Leonard, consulting engineer, of San Francisco. The Lindgren-Hicks Co. is the contractor.

Standard Topographical Signs.

The Board of Water Supply of New York has to make a great many topographical maps for copying by blue-printing. This drafting is done in a number of offices widely separated, and to insure uniformity in the work the symbols shown in the acompanying chart have been adopted for all maps on scales of 100 to 400 ft. to 1 in. For maps on a larger scale than 100 ft. to the inch, the symbols may be changed at the discretion of the draftsman. It will be noticed that these symbols are somewhat less detailed in some respects than those in most text books on topographical drawing. But two colors are used in addition to black, burnt sienna and blue; burnt sienna gives fairly good lines on blue prints, but blue gives poor lines, and for this reason blue used on tracings must be made opaque by backing with India ink or some other method that will insure printing. In reproducing the symbols by photo-engraving, it has been neces sary to reduce them from the original chart about three-tenths. So many inquiries for these symbols have been received by the Board that they are published here, by the consent of the Board, in order to be generally available for engineers interested in such work.



The Ratio of Heating Surface to Grate Surface as a Factor in Power Plant Design.

A paper read before the American Institute of Electrical Engineers on Dec. 19 by Walter S. Finlay, Jr.

Power plant design, in its modern development, is controlled solely by the specific application of general laws modified and moulded to suit special requirements. To attempt the construction of a comprehensive ruling from the results of a particular line of investigation in some particular plant, and then to advise the general use of such ruling as conducive to economical operation, would cause confusion, possibly resulting in a wholesale rejection of the good with the bad. The value of specific results and their publication lies in the opening up of a line of technical thought, or in adding information to some subject, from which specific deductions or particular application may be made.

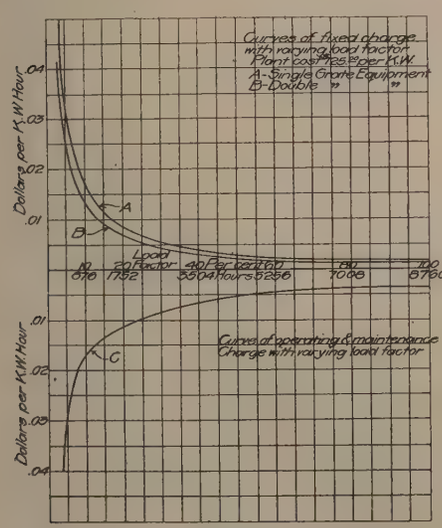

Curves of fixed charge with varying load factor
Plant cost $125 per KW
A-Single Grate Equipment
B-Double " "
Dollars per K.W. Hour
Load
1% 20 Per cent 40 60 80 100
876 1752 3504 Hours 5256 7008 8760
Curve of operating & maintenance Charge with varying load factor
C

Fig. 1. Fixed Charges; $125 Base.

The results obtained in the investigation which was primarily the foundation of this paper should be looked upon merely as specific, but whose bearing upon the general subject by means of a general development may be of value, particularly in certain new phases of plant design.

As a fundamental and almost initial point of attack in the comprehensive subject of steam power-plant design, the ratio of heating surface to grate surface has been a value fixed from the beginning of results of commercial usage, and the expression of the same in empyrical formulas or figures suited to the requirements of this or that designer, builder or manufacturer.

A summation of practice from early engineering times to values developed by the most modern idea, gives a great range to this ratio; namely, from the extreme value, as advised by Dalton in 1839, of ten to one, to modern values up to seventy to one used, not only in locomotive but even in power-plant practice. Of course, the primary object in view has been the adaptation of values to produce the maximum useful effect; but the question now arises as to whether or not "maximum useful effect" is not being interpreted as maximum economical efficiency, with reference to fuel only, as a primary consideration, and with an undue subordination of total plant costs. By total plant costs are meant, of course, the combined fixed charges resulting from interest on plant investment, depreciation, taxes, etc., and operation and maintenance charges.

Properly to investigate the subject in its particular applications would require an extremely tedious and complicated study of innumerable individual requirements; but for a general survey assumptions based upon commonly accepted values would suffice to direct the attention to the point involved.

Assuming a plant first cost of $125 per kilowatt, equipment, including turbo-generators, boilers equipped with stokers, with, say, sixty to one ratio, the following relative costs may be assumed:

	Per kw.	Per cent.
Total cost	$125.00	100.0
Building	43.75	35.0
Boilers	6.875	5.5
Grates	1.75	1.4
Piping	5.625	4.5
Coal-handling app. per kilowatt	2.30	1.84
Balance of equipment	64.70	....

The value of the building as assumed might be considered low, particularly in the case of a turbine plant; boiler cost is possibly average; grates, high—a stoker valuation; piping value is about average.

Assuming as a fair value for determining fixed charges: Interest on investment, 5 per cent.; depreciation, 6 per cent.; taxes and insurance, 1 per cent; then the total fixed-charge rate would equal 12 per cent.

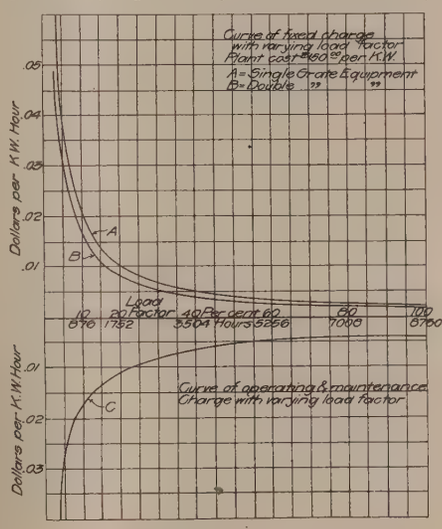

Curve of fixed charge with varying load factor
Plant cost $150 per KW
A= Single Grate Equipment
B= Double " "
Dollars per K.W. Hour
Load
1% 20 Per cent 40 60 80 100
876 1752 3504 Hours 5256 7008 8760
Curve of operating & maintenance Charge with varying load factor
C

Fig. 2. Fixed Charges; $150 Base.

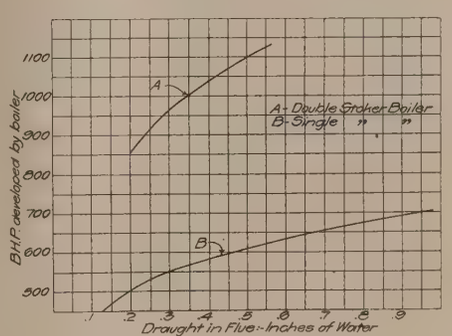


Fig. 3. Draft and Capacity.

Upon bases of load-factor and charges, a curve has been drawn showing the relative value of the fixed charges over a range of factor variation from approximately 3 to 100 per cent. in the case of the plant as assumed. (Curve A, Fig. 1.)

To determine total charges, the variation of maintenance and operation charges, relative to load-factor, must also be considered. It is rather difficult to assume this curve, as conditions in this respect vary rather widely. However, Curve C, Fig. 1, has been drawn through points located by comparative results obtained in actual cases of operation. The shape of the curve will practically be constant for any figure which may be assumed, and its relation to the general results will be such that the value of the principle involved will be unaffected. The sum of the ordinates between the two curves gives total charges per kilowatt-hour.

In a reconsideration of the plant design as affecting first cost, the natural method of procedure is to consider separately each item involved.

1. *Building.* In turbo-generator plant design it is a generally accepted rule that total plant dimensions are controlled by boiler-room dimensions; that, for instance, a diminution in the actual size of the boiler room may be accompanied by a proportional diminution in the size of the turbine-room, the output remaining the same. The methods of accomplishing such results are perhaps various; change in size of units, difference in type, closer grouping of units, etc.

2. *Boilers.* The consideration of this feature is naturally interlinked with the subject of "Grates," and the two can better be discussed together.

Rules of boiler-practice have been derived chiefly, if not entirely, by experiment and investigation; and those rules validated only by general acceptance can be quoted as bases for argument. Such a law is the following one:

All other conditions remaining constant, capacity developed is, with slight modifications, in direct ratio to the area of the active grate surface. An increase in capacity—heating surface remaining constant—caused by an increase in grate area, is accompanied by a loss in economical evaporation, due to the increased temperature of the escaping gases.

This loss in economy is the fundamental factor which must necessarily be the object of a careful study, involving the complete investigation of the heat interchanges taking place in a boiler. The research work of such men as Newton, Péclet, Joule and Rankine, together with recent investigation, has not, as yet, produced sufficiently definite and authoritative results, which may be used as bases of rational calculation in this regard. Under normal conditions of present boiler practice, estimates of loss vary from practically zero to as much as 15 per cent. fuel economy for an increase of 100 per cent. in boiler capacity.

Lately, however, the opinion has been advanced that considerable increase in capacity

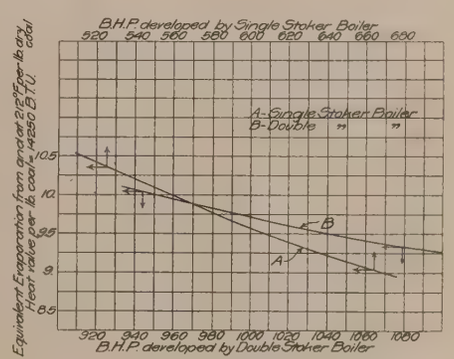

B.H.P. developed by Single Stoker Boiler
A-Single Stoker Boiler
B-Double " "
Equivalent Evaporation from and at 212°F per lb dry. Heat value per lb coal = 14250 B.T.U.
B.H.P. developed by Double Stoker Boiler

Fig. 4. Capacity and Evaporation.

can, without great sacrifice in economy, be obtained by proportional increase in grate area. This idea is based upon the possibilty that combustion and heat distribution and transfer could be much improved under the new conditions, when in increasing the grate area careful attention is given to details of design most conducive to these features. Other conditions being favorable, and with a belief in the correctness of this theory, a change was made in the design of eighteen of the boiler furnaces in the 59th St. plant of the Interborough Rapid Transit Co. (See Fig. 7.) Such a design gave the possibility of operating within the range of the original single-stoker boiler together with the higher range of the double stoker.

The second stoker installed, that is, the one beneath the mud-drum, as shown in Fig. 7, has an area of 80 per cent. of that of the original stoker. Certain features in the construction of the plant prevented installation of a larger size. A detailed description of points in the design is unnecessary, save to call attention to the fact that the lower stoker is constructed practically within a so-called "Dutch oven," and whatever is conducive to good combustion is provided for therein.

The curve shown on Fig. 8 is given as corroborating this fact. Operation of these stokers has shown that such is practically done with but little more complication than existed in the single type.

Tests to determine the comparative economical operation of the double and single stokers show results given graphically in Figs. 3, 4, 5 and 6. The curves in Figs. 3 and 4 are self-explanatory. Fig. 5 shows a curve plotted upon the values, as determined by curves in Fig. 4, this curve being indicative of the fact that operation with the double type was as economical in fuel values, as the single type increased in boiler capacity approximately 71 per cent. In the curves in Fig. 6 is shown the fact that the economical loss for an increase in rating of 80 per cent., as proportioned to the increase in grate area, varied between 2 and 3 per cent.

To summarize the results of these tests: It has been made evident that in this particular case double-stoker operation covers the entire range of single-stoker operation and adds an increase

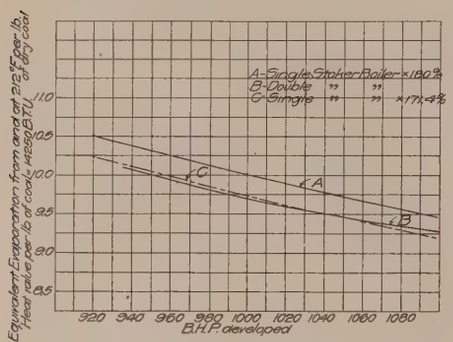

A-Single Stoker Boiler-×180%
B-Double " " "
C-Single " " ×171.4%
Equivalent Evaporation from and at 212°F per lb. Heat value per lb. of coal, 14,650 B.T.U.
B.H.P. developed

Fig. 6. Evaporation and Capacity.

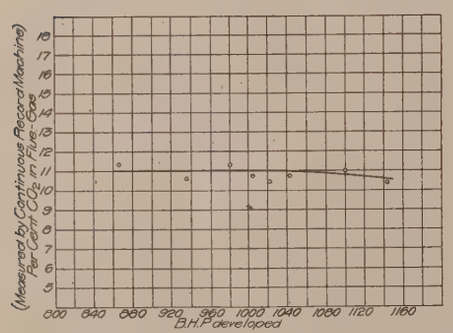


Fig. 8. Curve of Carbon Dioxide.

of capacity proportionate to its larger grate surface, with but slight loss in economy, and that the increase of 71 per cent. in capacity was accomplished with no loss in economy.

With these results as a basis, let it be assumed that boiler caapcity is increased in ratio to increase in grate surface with but little loss of economy. This view might be further strengthened when consideration is taken of the possibilities of economizer practice, the increase in saving, by proper design, being high in ratio to extra cost involved.

To return to the consideration of the items under the power plant whose first cost has been assumed to be $125 per kilowatt, the next point is:

3. *Piping.* In the case involved the cost of steam piping between boilers and manifolds, plus boiler feed piping, plus boiler blow-down piping, has alone been considered. With any change in number of boilers, capacity remaining the same, the cost of piping will vary in the same ratio times a factor due to change in size of pipe.

4. *Coal-handling apparatus.* Fixed plant capacity would seem to demand fixed cost of coal-handling apparatus, but the proportionate value of the conveying apparatus is so large that when any change is made affecting the length of carry the total system cost will be raised or lowered, although not in direct ratio to such change.

5. *Effect of change of ratio.* Suppose that in a reconsideration of the plant design it is decided to cut in half the ratio of heating surface to grate surface by the use of double grates or stokers under boilers of the same rating. Plant

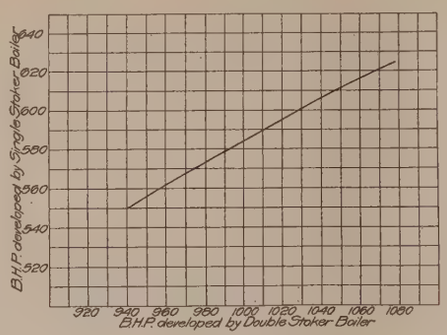


Fig. 5. Stoker Relations.

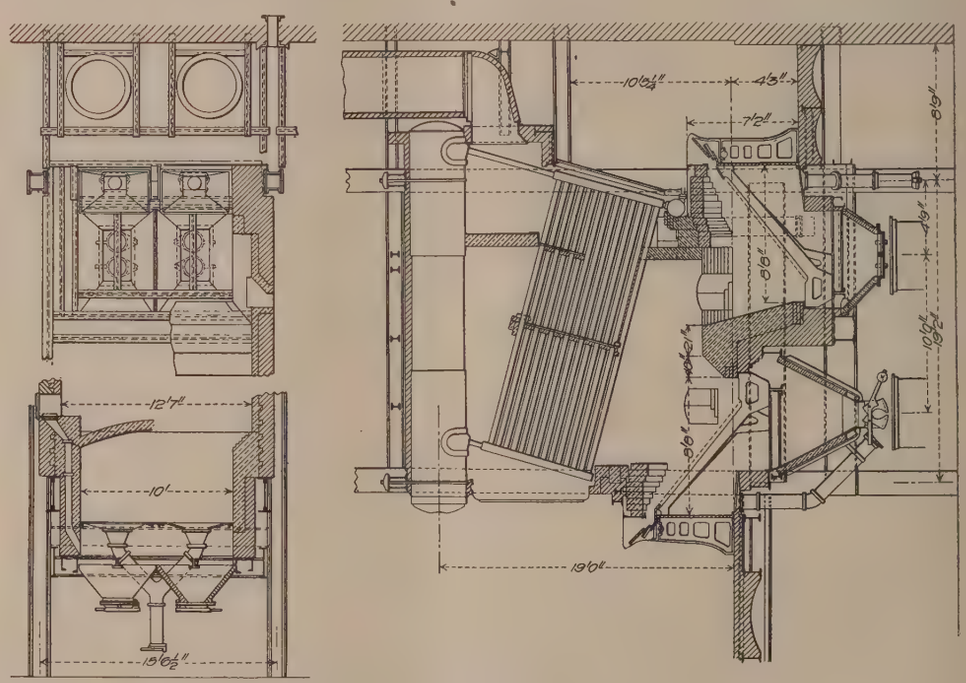


Fig. 7. Double Stoker Installation, Interborough Rapid Transit Co.

Water	0.90	0.90
Engine-room mechanical labor	1.70	1.70
Lubrication	0.44	0.44
Waste, etc.	0.38	0.38
Electrical labor	3.16	3.16
Total	100.00%	99.75%

The saving in boiler room maintenance and operation may be accounted for in the following itemized statement of boiler-room charges:

	Single Grate.	Double Grate.
Maintenance.		
Boilers	29.5%	14.75%
Economizers	2.78	2.78
Furnaces	17.29	17.29
Stokers and stoker engines	40.68	40.68
Boiler feed-pumps	5.42	5.42
Boiler feed-piping	2.20	1.10
Boiler blow-off piping	0.44	0.44
Water supply piping	1.52	1.52
Total	100.00%	83.98%
Operation.		
Water-tenders	20.82%	10.41%
Stoker operators	38.09	38.09
Assistant stoker operators	15.49	15.49
Stoker oilers	2.54	2.54
Economizer oilers and cleaners	5.84	2.92
Boiler feed-pump men	5.08	5.08
Boiler cleaners	10.41	5.21
Miscellaneous labor	1.73	1.73
Total	100.00%	81.47%

output is to remain the same. A tabulation of the costs as revised would be as follows:

	Per kw.
Building (reduced 40%)	$26.25
Boilers (reduced 50%)	3.438
tokers (remain same)	1.75
Piping (reduced 40%)	3.735
Coal-handling app. (reduced 15%)	1.955
Balance (remains same)	64.70
	$101.468

A curve (B, Fig. 1) is plotted upon this new basis.

Summation: Plant first cost and fixed charges, each reduced 19.6 per cent.

The next consideration is that of the effect of such changes upon plant maintenance and operation charges. Properly to discuss this, the following tabulation, based upon the figures given by Mr. H. G. Stott in his paper on "Power Plant Economics," will furnish a means of comparison:

Maintenance.	Single Grate.	Double Grate.
Engine room mechanical	0.64%	0.65%
Boiler room	5.40—(16%) =	4.54
Coal- and ash-handling app.	0.68	0.68
Electrical apparatus	1.41	1.41
Operation.		
Coal- and ash-handling labor	2.65	2.65
Removal of ashes	1.18	1.18
Dock rental	0.93	0.93
Boiler-room labor	8.38—(18.5%) =	6.83
Boiler-room oil, waste, etc.	0.21	0.21
Coal	71.94+(3%) =	74.10

Thus, with changes as noted, the decrease in maintenance and operation would be 0.25 per cent, the curve for same practically coinciding with Curve C, Fig. 1.

A second set of curves, based upon a plant cost of $150 per kilowatt, as shown in Fig. 2:

	Per kw.	Per cent.
Plant cost per kilowatt	$150.00	100
Building	60.00	40
Boilers	8.25	5.5
Stokers	2.25	1.5
Piping	6.75	4.5
Coal-handling apparatus	2.63	1.75
Balance	70.12	

Same plant double stoker.	
Building (reduced (40%)	$36.00
Boilers (reduced 50%)	4.13
Stokers (remain same)	2.25
Piping (reduced 40%)	4.05
Coal-handling apparatus (reduced 15%)	2.24
Balance	70.12
Total	$118.79

Showing a reduction in first cost and fixed charges of 20.8 per cent.

Summary. In the case of the $125 plant the following savings might be effected by use of double grate:

First cost, 19.6 per cent. saving.

Total plant charges varying from a saving of 5.64 per cent. at 100 per cent. load-factor to 7.54 per cent. at 50 per cent. factor to 9.65 per

cent at 4.16 per cent. factor (365 hours per year).

In the case of the $150 plant:

First cost, 20.8 per cent. saving.

Total plant charges vary from about 7.06 per cent. saving at 100 per cent. load-factor, to 9.26 per cent. at 50 per cent. factor to 11.5- at 4.16 per cent. factor.

Thus summarized, the remarkable effect that the grate area and heating surface ratio, when furance design is carefully considered, may have upon plant first cost and total annual costs should certainly place this particular feature well up in the list of subjects for careful investigation and make it a point of primary and fundamental consideration in advanced design.

Hydraulic Properties of Reground Cement Mortars.

A paper presented at the annual convention of the Association of American Portland Cement Manufacturers by Messrs. Henry S. Spackmand and Robert W. Lesley.

Observing in the course of our microscopic examination of cement mortars that a large proportion of the cement was unacted upon by water even after submergence for long periods, we decided to determine by actual test the extent of the hydraulic properties remaining in the cement after it had been gauged with water and allowed to harden. Having in the course of our regular routine cement testing, occasion to sample and test a large amount of one brand of cement, we determined to try the experiment on the broken neat briquettes from the seven and twenty-eight day tests of this cement.

The original cement, a well-known standard American brand, was sampled in carload lots, the sample being collected between July 17 and Sept. 16, 1905, and represented several thousand barrels. Each car was tested separately and all passed the requirements of the American Society for Testing Materials. In order to arrive at a standard of comparison, the results of the individual tests of the cars was averaged, giving the following as the mean of the different tests.

Fineness; 91.5 per cent. passed a No. 100 sieve and 76.2 per cent. passed a No. 200 sieve.

Setting time, as determined by Gilmore's needle; initial set, 170 minutes; final set, 334 minutes. Percentage of water, 21.

Soundness or constancy of volume; cold water pats, good; air pats, good; steam pats, good.

Specific gravity, 3.14.

Average tensile strength of standard briquettes; neat, 710 lb. at 7 days and 778 lb. at 28 days; 1:3 mortar, 261 lb. at 7 days and 360 lb. at 28 days.

As a further check an average sample was prepared from the various samples collected taking the same amount of cement from each sample and mixing thoroughly. The sample so prepared was tested in accordance with the standard specifications and gave the following result: Silica, 21.25; alumina and iron oxide, 11.09; lime, 62.96; magnesia, 2.35; sulphuric anhydride, 1.23; loss on ignition, 74; total, 99.12.

TENSILE TEST, STANDARD BRIQUETTES (AVERAGE OF 5 BRIQUETTES.)

Neat.		1-Cement, 3-Sand.	
7 days	610 lbs.	7 days	182 lbs.
28 days	752 "	28 days	338 "
3 months	754 "	3 months	487 "
6 months	646 "	6 months	415 "
9 months	593 "	9 months	350 "
1 year	706 "	1 year	322 "
2 years	670 "	2 years	325 "

The lower tensile tests obtained at the seven-day period can be accounted for by the cement having been exposed to air and aged considerably before the average sample was made up. For this reason, no test for setting time or specific gravity was made on the average sample.

In addition to the standard sieving tests the percentage of flour in the sample was determined by elutriation.

Percentage of cement passing 100-mesh sieve, 91.40.

Percentage of cement passing 200-mesh sieve, 76.30.

Percentage of cement remaining in suspension in dry kerosene after 30 seconds settling, 45.18.

Percentage of cement remaining in suspension in dry kerosene after 2 minutes settling, 29.85.

The broken neat seven and twenty-eight day briquettes made from the individual samples were saved, being stored in air in the cellar. After a sufficient quantity had accumulated they were dried at a temperature of 212° F. until of constant weight, then crushed and reground in a laboratory tube mill. The resultant product when tested as cement in accordance with the standard specifications gave the following results:

Fineness; 94.4 per cent. passed a No. 100 sieve; 74.4 per cent. passed a No. 200 sieve.

Setting time as determined by a Gilmore's needle; initial set, 60 minutes; final set, 310 minutes. Percentage of water, 25. Temperature of water, 62° F. Temperature of air 66° F.

Specific gravity, 2.57.

Soundness or constancy of volume; cold water pat, good; air pat, good; steam pat, good.

TENSILE TEST, STANDARD BRIQUETTES (AVERAGE OF 5 BRIQUETTES.)

Neat.		1-Cement, 3-Sand.	
7 days	123 lbs.	7 days	26 lbs.
28 days	253 "	28 days	63 "
2 months	340 "	2 months	145 "
3 months	383 "	3 months	179 "
4 months	402 "	4 months	227 "
1 year	401 "	1 year	198 "

The neat briquettes from the above test were in turn kept, being stored in air of cellar. On completion of the test, these were dried, reground and tested in the same manner as before, but the resultant cement was in addition analyzed. The results of the analysis after being twice gauged with water, were: Silica, 17.60; alumina, 5.67; iron oxide, 2.29; lime, 49.64; magnesia, 1.66; sulphuric anhydride, 1.13; water, 17.90; carbon dioxide, 3.84 = 8.27 $CaCO_3$.

The physical tests after three gaugings with water were as follows:

Fineness, 93.8 per cent. passed a No. 100 sieve and 73.6 per cent. passed a No. 200 sieve.

Setting time determined with a Gilmore's needle; initial set, 240 minutes; final set, 24 hours. Percentage of water, 26. Temperature of air, 73° F. Temperature of water, 70° F.

Soundness or constancy of volume; cold water pats, good; air pats, good; boiling pats, good.

Specific gravity, 2.35.

TENSILE TESTS, STANDARD BRIQUETTES (AVERAGE OF 3 BRIQUETTES.)

Neat.		1-Cement, 3-Sand.	
7 days	71 lbs.	7 days	17 lbs.
28 days	163 "	28 days	73 "
2 months	182 "	2 months	82 "
3 months	220 "	3 months	118 "
4 months	235 "	4 months	132 "
5 months	250 "	5 months	148 "

To confirm these results a similar test was made on another brand under conditions in all respects identical, with the following results:

Fineness: 96.3 per cent. passed No. 100 sieve, and 80 per cent. passed No. 200 sieve.

Setting time by Gilmore's needle: Initial set, 133 minutes; final set, 377 minutes. Percentage of water, 25. Temperature of water, 62° F. Temperature of air, 66° F.

Soundness or constancy of volume; coal water pat, good; air pat, good; steam pat, good.

TENSILE STRENGTH WITH STANDARD BRIQUETTES.

Neat.		1-Cement, 3-Sand.	
7 days	735 lbs.	7 days	265 lbs.
28 days	837 "	8 days	407 "

The test results obtained after regrinding the neat briquettes from the above cement were as follows:

The analysis of cement after the first gauging was: Silica, 19.88; alumina, 4.53; iron oxide, 2.03; lime, 55.30; magnesia, 2.64; sulphuric anhydride, .89; carbon dioxide, .68; water, 13.68.

Fineness: 97.1 per cent. passed a No. 100 sieve and 78.5 per cent. passed a No. 200 sieve.

Setting time by Gilmore's needle: Initial set, 209 minutes; final set, 434 minutes.

Soundness: Cold water pat, good; air pat, good; boiling pat, good.

Specific gravity, 2.56.

TENSILE TESTS, STANDARD BRIQUETTES (AVERAGE OF 5 BRIQUETTES.)

Neat.		1-Cement, 3-Sand.	
7 days	192 lbs.	7 days	57 lbs.
28 days	343 "	28 days	181 "
2 months	365 "	2 months	238 "
3 months	405 "	3 months	278 "
4 months	419 "	4 months	281 "
5 months	382 "	5 months	293 "
6 months	440 "	6 months	322 "
7 months	...	7 months	...

These experiments clearly show that even after cement has been twice gauged with water and allowed to harden under water, that all the cementing and hydraulic qualities are not destroyed, and that gauging with water and submergence in water does not retard the setting time of the reground cement as much as would be expected. Indeed, in the first test the setting time of the cement on being reground was quicker than in the original sample. A third conclusion is that it is only the very fine flour in the cement that is in condition to react when gauged with water and give strength to the mortars.

This last conclusion is further confirmed by other investigations undertaken at the same time to determine how fine the clinker had to be ground in order to be available for cementing action. Portland cement which had passed the 200-mesh sieve was further separated by elutriation into the following parts: (A) Material that settled out in 30 seconds. (B) Material that remained in suspension for 30 seconds, but settled out in one minute; (C) Material that remained in suspension for more than one minute. The cement thus divided into three portions according to size was treated with water in tightly stoppered tubes. "A" was only slightly acted upon by water, even after two years contact with it. "B" was only acted upon by water after three or four months, and only a portion became fully hydrated. "C" was acted upon almost immediately, swelling up and forming a very voluminous jelly.

Microscopic examinations of thin sections of neat Portland cement also show a large percentage of the cement unacted upon by water beyond a superficial or skin action. This being the case, then, in the regrinding of the neat briquettes after they had become hydrated, some of the coarser particles of the unacted-upon cement were reduced, by regrinding, to such a fineness as to become active. These three methods of investigation (first, microscopic study of set cement; second, the treatment of the various sizes of grains separated by elutriation; third, the regrinding of the neat cement after gauging with water and being allowed to harden under water), all lead to the same conclusion, that is, that only a comparatively small percentage of the original cement is ever acted upon by the water used in making the mortar, and that the portion of the cement which is acted upon is only the very fine impalpable flour which is not measured by present tests using sieves.

When cements were all ground in practically the same manner and on the same type of machinery, the use of sieves gave a fair indication of the flour present in the cement, it being a reasonable assumption that the finer the cement, the more flour present; but such is no longer the case with diversified methods of grinding and we have found more flour in cement ground on the burr stone showing a very poor degree of fineness as measured by the sieves than in some cements showing a very great fineness as measured by the 100-mesh and 200-mesh sieve. The fact that the sieving tests do not determine accurately the percentage of flour has been recognized in Europe, and various devices are being experi-

mented with to determine the percentage of flour. These investigations also show conclusively that all commercial cements contain a large amount of inert material and that the cement manufacturer is quarrying, grinding, burning, grinding again and paying freight upon from 50 to 60 per cent. of inert material which could as well be replaced by sand. Under present mechanical conditions it is commercially impossible to grind much finer than we are now doing, but one of the economies of the future in cement manufacture will be brought about by the perfection of grinding mechinery that will avoid the waste above mentioned.

A study of the chemical analysis of the reground cement is also interesting. Taking up the first sample and disregarding the presence of calcium aluminate and arbitrarily assuming that the silica was present in the original cement as tricalcium silicate, and that, on hydrating, it is decomposed to the hydrated mono-calcium silicate (CaO, SiO_2, $3H_2O$) and calcium hydrate, and neglecting the calcium sulphate but deducting the amount of lime present as carbonate, the combined water will account for about 70 per cent. of the cement having been acted upon by water, after being twice gauged and once reground, leaving 30 per cent. still present and capable of developing hydraulic qualities on being ground sufficiently fine, as is proven by the tests obtained on the second regrinding.

These figures are, of course, only approximate, since they do not take into consideration the lime present as aluminate and it is not positive that the hydration takes place as indicated above, but they are borne out in a general way by the results of the physical tests, for on regauging the cement developed considerable hydraulic qualities.

The analysis of the second sample of cement shows, on the same arbitrary assumption, after the first gauging with water, the hydration of about 45 per cent. of the cement, leaving 55 per cent. unacted upon.

In a recent article in "The Engineer" of Lonzdon, by Mr. W. Lawrence Gadd, entitled "Notes on the Le Chatelier Tests," he states in reference to Portland cement made by rotary kiln clinker that this "does not improve on aeration, but on the contrary almost invariably becomes more expansive, the increase being from 3 to 12 mm. after aeration for seven days; and many samples which, tested fresh, gave an expansion of only 2 to 5 mm., if again tested after seven days' aeration, would be condemned by the conditions of the British Standard Specifications as unsound. On the other hand, if the cement be kept for some time in a stoppered bottle or other closed receptacle, the expansion becomes less marked, and if kept long enough without exposure to air will become nil."

He explains this behavior of rotary kiln cement in Le Chatelier's tests as follows: "It is well known that rotary clinker is harder and therefore more difficult to grind than a chamber kiln clinker, and the ground product from the rotary kiln may be roughly divided into (a) flour; (b) comparatively large pieces of clinker grit, such as is retained on the 100 or 180-mesh sieve; (c) fine particles which are small enough to pass the 180-mesh sieve but are nevertheless particles of hard clinker grit. Tests show that although the chemical composition of this fine grit is practically identical with that of the flour, the former gives high expansion figures, whilst the latter gives rise to only slight divergence of the needles.

"The expansion appears to be as the proportion of fine grit, indirectly shown by the percentage of flour in the cement. Thus, to take an actual example: The respective products of two grinding mills were separately taken, carefully sampled, and tested by the Le Chatelier method. One of these mills (No. 1) was known to produce more flour and relatively less fine grit than did the other (No. 2). Both mills were concurrently fed with clinker from the same hopper, so that there should be no question of difference in composition or degree of calcination. The expansion results were: Cement from Mill No. 1, 3 mm.; cement from Mill No. 2, 17 mm.

"The fineness of the grinding in these two samples was practically the same, the residue on the 180-mesh sieve being in the one case 10 per cent. and in the other 10.5 per cent.

"The addition or removal of comparatively large pieces of clinker grit appear to have but little effect on the Le Chatelier test, and the addition of fine grit in the form of ground flints or sand has no appreciable action.

"It follows that the expansion, in the cases quoted, is due to changes in the fine particles of clinker grit under the influence of boiling water; and the view I wish to put forward is as follows: (1) At the time of gauging the cement the particles of fine flour, so called, are hydrated more or less completely. (2) Particles of clinker grit of comparatively large size are not hydrated, except, perhaps, on the surface, and act throughout as inert material. (3) Particles of fine grit clinker which are small enough to pass the 180-mesh sieve are only slightly hydrated at the time of gauging. The cement then sets hard, but under the influence of heat, when the block is boiled, the hydration of these small particles continues, with consequent expansion.

"The increase in expansion, after the cement has been subjected to aeration, may perhaps be explained by the assumption that the fine particles of clinker grit, on exposure to moist air, commence to disintegrate to some extent, and are thus rendered more easily hydrated when submitted to boiling heat. That something of this nature does happen is rather borne out by the fact that rotary clinker which has been weathered by exposure to air for some time is more easily ground, than if taken direct from the kiln, and such weathered clinker invariably gives lower expansion figures, provided of course, that it was properly burned, and contains no underburned portions."

While the behavior of the cement by the Le Chatelier tests for soundness is of little interest to us, however, as this test is not used in this country, of interest to us is the explanation offered by Gadd for the behavior of the cement, since this explanation is in line with the conclusions drawn in this article, that is, that the fine flour of cement is the only portion which is acted upon by water, hence comprises the portion of cement which gives to the mass its hydrated qualities.

Book Notes.

The exceptional value of the work done by the American Society for Testing Materials is clearly shown in the "Proceedings of the Tenth Annual Meeting," which has just appeared. It is a volume of 759 pages filled with information of the most important character for all engineers who have to do with the design of structures or machinery. This journal has held the opinion for a number of years that no engineer who has much designing to do can afford to be without a set of these volumes. They not only contain the latest ideas of thoroughly representative committees concerning the all-important subject of specifications, but they also cover all advances in testing materials and progress in research work to ascertain the strength of materials and standard products constructed from them, such as rails, castings, car wheels, reinforced concrete columns and beams, and the like. The object of the Society is, briefly stated, to furnish information concerning materials and the best methods of using them. The field is manifestly a wide one, and as the Society covers it thoroughly, its publications are invaluable to those who desire to keep abreast of the times. The proceedings which are recorded in the volume just issued were reported in the Current News Supplement of this journal of June 29, and it is therefore unnecessary to review the contents of the volume at length in this place. (Philadelphia, Prof. Edgar Marburg, University of Pennsylvania, $5.00.)

The twenty-first annual number of the "Mechanical World Pocket Diary" is an astonishing example of book making. It has nearly 400 pages, is bound in cloth, and yet it retails in Great Britain for only 12 cents. The diary gives ample space under each date to jot down all the notes that the average engineer keeps in such a book, while the mechanical information is comprehensive and accurate. This technical section has now assumed a comparatively permanent form, and the changes made from year to year are mainly to bring the contents up-to-date in those departments where new standards have been adopted or the progress of the art makes revision necessary. While the contents are by no means so complete as the information given in more elaborate enginering handbooks, there are few things which arise in the general run of mechanical drafting room work that cannot be answered at least in part, by reference to this little volume. (Manchester, England, Emmott & Co., 65 King St., 6 pence.)

A little book, professedly written for the engineer and engine man, but useful for a much wider circle of readers, is the "Engine-room Chemistry" of Prof. Augustus H. Gill, of the Massachusetts Institute of Technology. It aims to give such information concerning oils, fuels and the phenomena of combustion and steam-raising, all expressed in fairly non-technical language, as will enable the average reader to understand what is necessary from the point of view of chemistry to obtain the maximum efficiency in the power plant. The book opens with an explanation of those elements of chemistry which must be understood in order to study problems of this nature and describes the apparatus and chemicals used in making the tests under consideration. The author then takes up in detail fuels and their analysis, and gives in the next chapter an explanation of methods of regulating combustion and conducting gas analyses. Boiler scale, fitting and corrosion form the subject of the following chapter, in which the rather complicated subject of hard waters is outlined in a satisfactory manner. It is rather unfortunate, however, that this chapter was written before the presentation of the important paper by Dr. Cushman explaining the method of preventing pitting which is being followed with much success in some boiler plants in Washington. The remainder of the book is devoted to mineral, animal and vegetable oil and will be found particularly useful to those who have learned by experience the importance of more attention to this class of engine room supplies than is usually paid it. (New York, Hill Publishing Co., $1.00.)

Although there are many books on mechanical drawings available for self-instruction and classroom work, a new one entitled "Structural Drawing," by Mr. C. Franklin Edminster, of the Pratt Institute in Brooklyn, takes up a branch of the subject which has hitherto not been touched except very briefly. It gives a course of lessons in the drafting work in structural shops and the offices of architects and mill engineers, so far as steelwork is concerned. The book opens with the usual instructions concerning drafting materials, which are followed by a collection of geo-

metrical problems. Simple projection such as is used in preparing working drawings is then explained, after which structural drafting is taken up in detail. The first work is the drawing of cross sections of rolled shapes to scale and their lettering according to standard dimensions; the drawing to scale of nuts and bolts, tie rods, eyebars and clevices, turn-buckles and anchors is then taken up and the more elementary connections are introduced. Afterward the student is taken along step by step as his skill increases until the most difficult forms of structural drafting connected with staircase details have been taken up. The many plates in the book are executed in the standard manner required in bridge shops, and the book as a whole is an excellent outline of the somewhat special class of drafting necessary for steel. (New York, David Williams Co., $2.50.)

The long-awaited "Municipal and Private Operations of Public Utility," the report to the National Civic Federation by its Commission on Public Ownership and Operation, has at last appeared in three bulky volumes, forming the latest and most complete word on this important subject. The first volume contains the final conclusions of the Committee and general summaries of the evidence upon the various phases of the question. These conclusions were reviewed at considerable length in The Engineering Record several months ago. The second volume is devoted to the reports of the experts upon conditions found in the United States, and the third volume is devoted to the situation in Great Britain. These volumes are made up of two entirely distinct parts. The first is a collection of information regarding public service undertaking, compiled by specialists in such work and approved as correct by the members of the committee who believed in municipal ownership and by those who believed in private ownership. The second portion of the volumes is a collection of monographs based on the data and written from the viewpoint of each interest. The reader desirous of making a thorough study of the relative advantages of municipal and private ownership will find the volumes of particular value on this account, for the advocates of each method of operation are here compelled to base their arguments on exactly the same basic facts. This is probably the first time that such a complete discussion of the subject has been presented under conditions so favorable for clearing away the unessential assumptions and disputed facts that make the usual article on the subject hardly worth reading. Inasmuch as the report has already been reviewed in this journal and an estimate of its value stated editorially it is hardly necessary to say more in this place than that the volumes should be in every library claiming to possess the more important works on the control of public service utilities. (New York, 281 Fourth Ave., National Civic Federation, $10.)

Letters to the Editor.

STRESSES ON RIVETS.

SIR: I notice that, while your standard specifications provide carefully for the effects of repeated stress in the members of a framework or structure, the effect on the rivets and pins is practically ignored, and constant values are assigned for safe shearing and bearing stresses on these. As a chief draftsman and designer of steel structures, I use the following formulas, which ensure that the connections are as strong as the pieces connected:

$$(1)\quad P = 9{,}000\left(1 + \frac{\text{minimum}}{2\ \text{maximum}}\right)$$

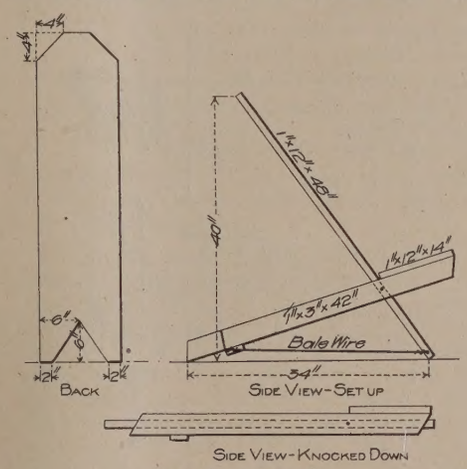
$$(2)\quad P = 9{,}000\left(1 - \frac{\text{minimum}}{2\ \text{maximum}}\right)$$

The first formula gives the unit stresses on mild steel rivets in single shear for repetition stresses, and the second gives the unit stresses for reversing loads. The safe bearing stresses and double shear stresses are double the amount given by these formulas. These formulas are original, so far as I know, and perhaps if you found space for them in The Engineering Record they would receive criticism and light would be thrown on the subject. Yours faithfully,

Bradford, Eng., Oct. 8. JAMES A ORREL.

CAMP COMFORTS.

SIR: When I arrived at the engineers' camp at Del Rio it was about as uninviting a place as I ever saw. The dirt floors of the tents consisted of an inch of dust—so fine that one could



A Camp Chair.

not stir without filling the tent with the powder. It seemed to me that the first night I spent there I breathed in my full "peck of dirt." This was intolerable and the next morning I had the men with our team get gravel from the river nearby and spread a layer of it 3 in. thick in each tent. We have had no dust since then. For seats we had boxes, and two roughly-made stools. Now, when I am working I work hard and when the time for rest comes I want to rest as thoroughly, which I cannot do on a tomato box nor very much better on a condensed milk box, so I determined to improve our sitting arrangements. The commissary had just received a quantity of 12-in. white pine boards for general camp use, and I ordered a half dozen of them sent to our camp. From these we made three easy chairs similar to those here shown. I had often seen these chairs in use, but always made rigid, and hence very unhandy to transport when moving camp. I therefore determined that the present lot should not have that defect. We first cut three pieces 4 ft. long for the backs, next six pieces 4 ft. 6 in. long by 3 in. wide, then three pieces 14 in. long and three 1 ft. 3 in. Placing the 4-ft. board on the ground, one of the 3x42-in. pieces was placed on edge on each side, and the pieces 14 in. long nailed crosswise to the 3-in. pieces. Turning the back board and sides over without disturbing their relative positions, one of the pieces 1 ft. long and 3 in. wide was nailed on 6 in. from the end opposite to where the seat was nailed. Next the back and side boards were set up as shown in sketch and a ten-penny wire nail driven in the center of the side piece just at the back of the seat and into the back piece 1 ft. from where it rests on the floor. The chair back can be fixed at any angle, but I found the angle shown to be the most satisfactory to all who tried it. The right slope of back board can be found by raising the top until it is 3 ft. 4 in. from the floor. To keep the chair in this position I used two bale wires, driving an eight-penny nail through the loop and into the bottom of the back piece and winding the other end around the 3-in. piece as shown. By unfastening the wire, the chair can be knocked down and occupies a space only 48 x 14 x 5 in.

For this work we had only a saw and hammer. We used fourteen nails and less than 8 sq. ft. of boards for each chair. I give this account of the work and sketch of the chair, not because I think I have made something new—I first saw this kind of chair thirty years ago—but because I have never seen a drawing of it, and I want to suggest to others to try it, and see if it is not one of the most comfortable a tired man ever threw himself into. HOWARD EGLESTON.

Del Rio, Mexico.

EXAMINATIONS FOR PHILIPPINE CIVIL ENGINEERS.

SIR: In an examination for civil engineer, in the Departmental and Phillipine Services, given by the United States Civil Service Commission at various points throughout the country on Oct. 16th and 17th, the subjects, with their weights, were as follows: (1) Pure and Applied Mathematics, "involving a fair knowledge of pure mathematics to and including calculus"; weight, 20. (2) Use and construction of instruments and surveying; transit, stadia, level, plane table, rod, chain, tape, current meter, etc.; weight, 30. (3) Design and construction, requiring knowledge of highways, railroads, dams, retaining walls, trusses, foundations, etc.; weight, 25. (4) Training and experience, rated from application form, answers to which are made under oath; weight, 25.

The time allowed for the examination was six hours on the first day and three hours on the second day, the first two subjects being given on the first day,

The questions asked, as nearly as I could recollect them the day after the examination, were as follows:

Mathematics; three questions to be answered. (1) In a hollow cast-iron column 16 in. in diameter the thickness of metal is 1½ in., allowable stress is 9,000 lb. per square inch, column 24 ft. 5 in. high. $W = w + (wln^2/r^2)$, in which l = length of column in inches, $n = .002$, r = radius of gyration, W = load on column, and I for cylindrical form = $11d^4/224$. Find w, in tons, to one decimal.

(2). The points, A, B and C, are in a straight line, $AB = 376.5$ ft., P is a point outside of the line. The angle $APB = 61° 20'$; and the angle $PAB = 42° 6'$. If D bisects ABC find angle ADP, to the nearest minute.

$$\frac{a - b}{a + b} = \tan \tfrac{1}{2}(A - B)\cot\frac{C}{2}$$

(3). In a triangular frame ABC, $AB = 15$ ft.; $BC = 12$ ft., and $AC = 20$ ft. There is a force of 6, normal to AB at A, one of 8 normal to BC at C, and a vertical one of 6 at B. The frame is fixed only at A. Find, by any method, the stresses in the parts of the frame.

(4). The bottom of a channel is 8 ft. wide, the water level is 16 ft. wide, the sides slope at 30° from the vertical, the flow is 288 cu. ft. per second and the grade 1 in 2,000. Find the mean depth, mean velocity and coefficient of friction.

Use and Construction of Instruments and Surveying; four questions to be answered.

(1). Name all of the ways errors can occur in the use of the transit, and state how to eliminate or reduce them.

(2) Give same for the wye level.

(3). State, in detail, how you would adjust the standards of a transit. What previous adjustment should be made and why?

(4). State, in detail, how you would proceed to make a topographic survey and map complete (*a*) of a very irregular plot of ground of not over 350 acres and showing 2-ft. contours to a large scale. (*b*) Give the same for a large similar plot on a small scale and showing 20-ft. contours.

(5) Give an account of the Government's method of surveying the public lands.

(6). Tell, in detail, how you would make a hydrographic survey of a river averaging 300 ft. wide and 15 ft. deep.

Design and Construction; three questions to be answered.

(1). Give a list of all the defects found in lumber and state the cause of each.

(2) Write as complete a specification as you can (*a*) for a brick pavement, (*b*) for an asphalt pavement.

(3). Given a town of 5,000 inhabitants. Describe in detail all the data you would collect before starting the design of a sewerage system.

(4). Given a reservoir of certain capacity. How would you determine the dimensions of dams, fills, walls, embankments, etc.? Show diagrams of same. How would you locate spillways and determine size of same?

(5). Give a list of the advantages and disadvantages of the different kinds of highways in use. Give sketches showing best methods of construction. COMPETITOR.

COST ANALYSIS DATA.

SIR: In compliance with your request of Dec. 5, I send you a brief outline of my opinion regarding the practicability of using cost-analysis data with safety in estimating the probable cost of constructing engineering works.

It cannot be disputed that a complete system of cost keeping, in sufficient detail, is of great value to public works departments. If the work is performed by day labor such a system will serve as a useful guide to the engineer in charge, both in making his estimates of the probable cost of work and in checking from time to time the actual cost of construction with the cost of similar work previously done. Such a system is of nearly as much value to departments conducting work by contract. The greatest value of cost keeping in all cases, of course, results from the costs having been kept upon work under the direct personal supervision of the engineer who is using the cost system as a guide for estimates and for checking costs during construction. These observations imply that the use of such cost statistics is confined in each case to one locality, in which similar classes of work must be reasonably comparable. In making an estimate of the probable cost of work under these conditions the judgment of the engineer is of the utmost importance, and even with a satisfactory cost system his judgment must far exceed in comparative value the weight given to his statistics of cost upon past work.

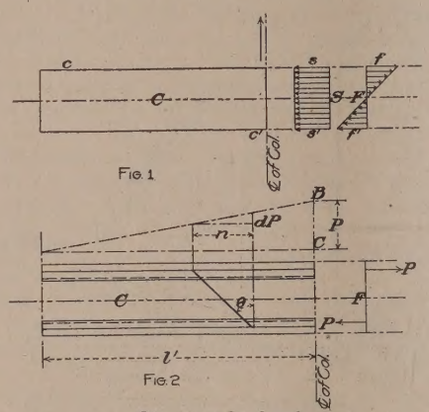
In the application of costs obtained by other engineers, or in other localities, the utmost care should be exercised, as it is impossible in almost all cases to have an accurate knowledge of the particular conditions surrounding the work done at the costs given. Such costs may serve as a very valuable general guide, but ought not to serve as the main basis of estimates, which should in every case be the result of personal judgment growing out of personal experience. If the engineer making the estimate has obtained the cost of other similar work under his own direction these costs would be of much greater value to him in assisting him to form his judgment than would be the costs obtained by other parties under conditions which must be largely unknown to him.

In fine, it is my opinion that cost statistics, as generally accessible from published records, are of very doubtful value for anything more than a general guide and may be exceedingly dangerous to follow. Very truly yours,

HARRISON P. EDDY.

Boston, Mass., 14 Beacon St.

[This letter is an answer to a request made to Mr. Eddy to send for publication in The Engineering Record a statement of his conclusions regarding the value of cost analysis data. As superintendent of the sewer department of Worcester, Mass., from 1892 to 1907, he constructed by day labor all kinds of sewerage works, involving ordinary work in rock and earth, heavy trenching, difficult pile driving, tunneling in many kinds of material, and concrete structures. All payments for materials and labor were made by him directly, and for a number of years he kept very complete cost analysis records of these expenditures and the conditions of the construction. Probably very few engineers have had such an opportunity to study actual cost data on sewerage work, and for this reason Mr. Eddy's opinion deserves especial consideration.—EDITOR.]



Stresses in Lacing.

STRESSES IN LACING ON COLUMNS.

SIR: One of the fundamental features of short column design is the assumption that the compressive stress is distributed uniformly over the area of any cross-section. Suppose *C* in Fig. 1 to be such a column. Then *S* may represent the stress diagram. Now suppose the column becomes longer so that there is bending, or a tendency to bend in the direction shown by the arrow. There will then be set up in the column a flexure couple that we assume will take the form shown as *F*. It will be evident at once that the maximum stress under the second condition will be greater in compression and smaller in tension. The maximum stress on the tension side will be $(f-s)$. The maximum stress on the compression side will be $(f'+s')$. We might concentrate the entire stress shown by diagrams *S* and *F* about points respectively the centers of gravity of the parts of the diagram above and below the neutral axis. The stress shown by diagram *S* is balanced about the neutral axis, and therefore would not have a tendency to distort the column laterally. Lateral distortion of the column would depend on the flexure couple only. It is then this couple that has to do with the design of lacing in built-up columns.

Fig. 2 is a column composed of two channels laced back to back. The flexure diagram *F* is shown with forces concentrated at the centers of gravity of the channels. For practical purposes we may assume that $P=Af$ where *A* is the area of cross-section of one channel and *f* is to be taken as the maximum stress in the channel due to flexure at the point where the greatest bending, or the tendency to bend, occurs. In most cases, as that shown in Fig. 2, this point will be at the center of the column. In any practical case *f* should be taken as the difference between the allowable stress for the material in a short column and the calculated unit stress for the column in question. For instance, when the straight-line formula is used we have $s=s'-cl/r$, where *s* is the stress for a short column, *c* a constant, *r* the least radius of gyration, *l* the equivalent length, and *s* the stress for a long column. Then, in our discussion, the value of *f* is cl/r.

When Euler's formula is used

$$s - \frac{EI\pi^2}{A'l^2} = f$$

Where A' is the area of column cross section, and *I* is the moment of inertia about an axis perpendicular to the plane of bending.

Returning to Fig. 2, l' is the distance from section of maximum to section of no flexure. When *l* is equal to the length of the column in question l' may be equal to ½, ¼, etc., according to end connections. Upon the line *AC* at the point *C* we erect a perpendicular *BC* which shall equal to scale $P=Af$. Now at the points *A* and *C* we know the flexural stress. If we were to calculate it at all other points between *A* and *C* and plot to scale along AC as a base there would result a curve similar to a harmonic curve. A circular arc would not be greatly different, and for all practical purposes a straight line drawn from *B* to *A* will be sufficiently near. Then when *n* is the interval for lace spacing there results d*P* the change in flexural stress.

Then (1): $dP \csc \Theta = G$,

Where *G* = stress in lace bar. From similar triangles $dP = nAf/l'$. Substituting in (1) this value of d*P* there results

(2) $G = nAf/l' \csc\Theta$. Where there is a double system of lacing it is consistently accurate to take one-half the computed value of *G* as the stress in each bar of the panel.

The same reasoning makes it easy to derive formulas for the stress in lace bars where there are any number of leaves in the column. For example, we may assume a column where there are four leaves. The maximum stress will occur between the inner two leaves.

Let d_1 = distance from the neutral axis to the center of gravity of first leaf, and d_2 = distance from the neutral axis to the center of gravity of the second leaf. Let A_1 and A_2 be the area of the inner and outer leaves respectively. Then

(3) $P = (d_1/d_2)\, fA_1 + fA_2$.

$$G = \frac{fn}{l'}\left(A_1\frac{d_1}{d_2} + A_2\right)\csc\Theta.$$

For the purpose of illustration assume a column made up of two 12-in., 20.5-lb. channels laced 13.75 in. out to out with square ends and an over all length of 24 ft. The area of a cross-section of this column is 12.06 sq. in. Assuming a factor of safety of 4, Gordon's formula gives a maximum safe unit stress of about 11,500 lb. Calling the safe stress for a short column of this steel 16,000 lb., there results f = 4,500 lb. Let $\Theta = 45°$.

$$G = \frac{10\times 6\times 4{,}500\sqrt{2}}{6\times 12} = 5{,}300 \text{ lb.}$$

This would give for a 2x¾-in. lace bar a stress of 10,600 lb. which it is safe to allow under these conditions. G. L. BILDERBECK.

East Berlin, Dec. 3.

PRIVATELY OWNED RAILROADS in Belgium will shortly have their total length reduced to 200 miles, as the Belgian Parliament has authorized the purchase of a privately owned line 112 miles long. The 200 miles referred to are operated by six different companies.